AN INVESTIGATION OF THE BOND BETWEEN PORTLAND CEMENT MORTAR AND COARSE AGGREGATE

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by

C. F. SCHOLER

Joint Highway Research Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

Final Report

AN INVESTIGATION OF THE BOND BETWEEN PORTLAND CEMENT MORTAR AND COARSE AGGREGATE

TO: K. B. Woods, Director
Joint Highway Research Project

December 17, 1964

FROM: H. L. Michael, Associate Director
Joint Highway Research Project

Project: C-36-42H
File: 5-9-8

Attached is a report entitled "An Investigation of the Bond Between Portland Cement Mortar and Coarse Aggregate." The report was prepared by Professor C. F. Scholer under the direction of Professor J. F. McLaughlin. Professor Scholer also used this report as his thesis for the Ph. D. degree.

Professor Scholer is now an Assistant Professor on the staff of the School of Civil Engineering in the field of highway materials.

Respectfully submitted,

Harold L. Michael

Harold L. Michael, Secretary

HLM:kr

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Charles F. Scholer

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Purdue University
Lafayette, Indiana
December 17, 1964

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ABSTRACT

Scholer, Charles Frey. Ph.D., Purdue University, January, 1965. An Investigation of the Bond Between Portland Cement Mortar and Coarse Aggregate. Major Professor: John F. McLaughlin.

This investigation was concerned with examining and explaining factors contributing toward bond between coarse aggregate and portland cement mortar. In addition, the influence of mortar-aggregate bond strength on concrete was measured by flexural and compression tests of concrete.

A method for measuring the mortar-aggregate bond was developed utilizing small cores approximately 0.6 inch in diameter and 2.0 inches in length. These specimens were cored from slabs which consisted of a one inch thick slice of rock bonded to a one inch thickness of portland cement mortar. The core specimens were broken, at the bond interface in a cantilever loading apparatus. Test specimens were not allowed to dry prior to testing. The test has a high variability as do other bond tests that have been used to measure mortar-aggregate bond. However, the method developed allows a large number of specimens to be prepared without undue effort and with a great deal of physical uniformity between the test specimens.

Concrete was prepared from some of the same aggregates whose mortar-aggregate bond strengths were measured. In order to minimize

variables other than bond strength, the aggregate was rounded with abrasives and one-size, 1 to 3/4 inch, coarse aggregate was used for the concrete.

All mortar in both the mortar-bond test specimens and in the concrete specimens had a water-cement ratio of 0.60 and a sand-cement ratio of 2.50, by weight. Graded Ottawa sand was used in the mortar throughout the investigation. The majority of the specimens were tested at an age of 14 days.

A variety of coarse aggregates was used including artificial aggregates such as glass and plastic. The latter were used to provide aggregates with low mortar-aggregate bond strengths. Significant differences in mortar-bond strength were found between different types of aggregates; however, the differences were not large among the natural aggregates. The highest bond strengths were obtained with quartzite.

No relationship was found between aggregate grain size, chemical properties or physical properties, determined in the investigation, and the mortar-aggregate bond strength. For the conditions of no drying (i.e., constant saturation) of the mortar-aggregate bond specimens in this investigation, the surface finish of the coarse aggregate did not usually affect the bond strength.

The concrete strengths were significantly influenced by the mortar-aggregate bond strength only if the bond strengths were greatly different (i.e., limestone, glass and plastic). Among the concretes made with natural aggregates there were not significant differences in strength.

Mortar-aggregate bond strengths are a complex combination of mechanical and chemical bonds. This investigation indicates that chemical bonding is responsible for a considerable amount of the bond developed between mortar and aggregates.

The occurrence of microcracking in the compression test specimens was determined by electronic pickups, amplification and counting of the impulses created by the cracking. The relationship of the occurrence of microcracking, the ultimate strength of the compressive test specimen and the mortar-aggregate bond strength of the coarse aggregate is discussed.

INTRODUCTION

Portland cement concrete may fracture in three different ways. The fracture may extend through the aggregate and the cement paste; this is common in high strength concrete. The fracture may occur at the interface of the aggregate and paste, and through the paste as it moves between interfaces. This is common in low strength concretes. The third feasible way for fracture to occur is through the paste alone, but this is not a common type of fracture. Of the three types, fracture at the interface of the aggregate and the paste has obvious implications of bond failure. The other common type of failure, that wherein fracture extends through the aggregate, does not clearly indicate the importance of bond. If one considers that initiation of fracture occurs at some weak point in the concrete and then proceeds on toward failure it is likely that bond plays an important part in concrete strength by producing weak points, even when the fractured surface does not show a large number of bond failures between the aggregate and the paste.

The bond between aggregate and paste or between the coarse aggregate and the mortar in a quality portland cement concrete has long been considered essential. The concept of bond has been widely used to explain many of the observed properties of concrete (1, 2, 3)*.

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*Numbers in parenthesis refer to references listed in the Bibliography.

The importance of bond was mostly intuitive until recently when several investigations have shown that it does indeed influence the strength of concrete (4, 5).

Several studies (1, 4, 5, 6) have been directed toward measurement of the strength of the cement paste-aggregate bond or the cement mortar-aggregate bond. These studies were in substantial agreement in at least one important result: different bond strengths were obtained for different aggregates. With regard to the actual nature of the bond, little work has been done to determine how it occurs and what chemical and physical properties contribute to the production of good or poor bond. This investigation was directed toward determining the nature of the mortar-aggregate bond and how it influences concrete strength.

Early investigations were concerned with the bond in masonry construction i.e., the bond between mortar and brick or stone. ASTM Designation: C 321, Test for Bond Strength of Chemical Resistant Mortars, was developed to measure such bonds. Concern for these bonds between mortar and building materials remain of importance. Recent work has been reported by Hoberg (7) who measured the bond between mortar and various building materials. His investigations indicated that a layer of paste was squeezed out between the mortar and a non-absorbent building material. This layer produces a better bond than if a drier mix or an absorbent building material were used.

Early measurements of mortar-aggregate bond, with particular reference to aggregates in concrete, was done using the briquet testing apparatus now described in ASTM Designation: C 190, Test for Tensile

Strength of Hydraulic Cement Mortar (8). This apparatus was recently used by Hsu (5) in determining direct tensile strength in connection with his studies of microcracks between coarse aggregate and paste mortars. An aggregate cube cast in a rectangular mold with cement paste bonded to each end produced a prism which could be tested in cantilever loading. This type of specimen and method of loading was used by Alexander and Wardlaw (1, 9) to obtain the modulus of rupture as a measure of the aggregate-cement bond.

A summary of the pertinent conclusions from previous investigations shows:

1. The mortar-aggregate bond strength varies with the type of aggregate (1, 4, 5, 6).
2. Surface roughness influences the mortar-aggregate bond strength (1, 5, 10).
3. The mortar-aggregate bond strength generally increases as the water-cement ratio decreases (1, 5).
4. The moisture conditions at the time of testing are critical, presumably due to drying shrinkage stresses (5).
5. The size of the aggregate surface bonded influences the bond strength. It was indicated that bond strength was proportional to the log of the reciprocal of the area bonded (9).
6. Aggregate-paste bond strength is an important factor of the strength of concrete in both flexure and compression. Alexander et al. (4), showed that the compressive strength of concrete cubes was proportional to the quantity (B+2P)

where B = the modulus of rupture of the cement-aggregate bond in psi and P = the modulus of rupture of paste.

For (B+2P) values of about 1,800 psi and 5,000 psi the compressive strength of concrete cubes were about 2,000 psi and 6,000 psi respectively. If a constant paste strength were used, it becomes evident that the lowest bond strength (modulus of rupture) reported (700 psi) might give a compressive strength of 5,200 psi compared to a compressive strength of 6,000 psi for the highest bond strength (1,500 psi) reported in the investigation.

The effect of bond on the flexural strength was shown by bond strength values of (700 psi) and (1,500 psi) resulting in flexural strengths for the concrete of 600 psi and 950 psi respectively.

7. Bond strength generally increases with age (1, 5).

The bond between mortar and aggregate has been considered to be of two types, mechanical and chemical. The mechanical bond, idealized as an interlocking of cement paste and aggregate, is supposedly enhanced by the rough-surfaced aggregates. The chemical bond is developed by one or more types of physico-chemical bonds such as ionic bonds and van der Waal bonds.

Farrand (11) was one of the first to investigate the cause of bonding between portland cement paste and aggregate. He studied the amount of impregnation that occurred in different minerals, impregnated with colored Canada balsam by making petrographic thin sections.

He tested a variety of minerals in tension and obtained various bond strengths and also studied the surfaces of crystals which had been in contact with cement paste. Farrand obtained the greatest bond strength with calcite and attributed it to what he called "corrosion" of the calcite. He attributed this "corrosion" with bonding to some mechanical strength, possibly some epitactic crystal growth at the paste-calcite interface and the formation of a solid solution between the calcite and the paste. He did not observe the "corrosion" with other minerals. His observations with calcite are of special interest considering the now known cement-carbonate rock reactions (11, 12).

Munger (6) investigated what was apparently a relationship between the cement paste-aggregate bond and the number of ionically bound oxygen atoms in the surface of a mineral aggregate. While many aggregates had paste-aggregate bond strengths higher than indicated by their oxygen ions the minimum values obtained show some correlation with the calculated number of effective oxygen ions per square micron and the tensile bond strength.

The existance of portland cement mortar-aggregate bond has frequently been used to help explain theories of concrete failure. Recent investigations in this field have studied the formation of very small internal cracks, called microcracks, in hardened concrete as it is loaded in compression (3, 5). These cracks can develop prior to loading (6), however they are more often formed when the concrete is under compressive load. Under compressive load microcracks appear at forty to sixty percent of the ultimate load, and continue to increase in

number until failure. It is believed that the formation of these microcracks is due to the tensile failure of the mortar-aggregate bond which is where they frequently occur (3, 5).

PURPOSE AND SCOPE

The purpose of this research was to, a) examine and explain the factors contributing toward bond between aggregate and portland cement mortar and, b) determine the influence of bond strength or bond intensity on the strengths (measured by different tests) of concrete.

There is no standard test for measuring mortar-aggregate bond nor has any one method been exclusively used in previous investigations involving measurements of bond strengths. The methods of bond measurements used by other investigators were considered and several procedures were experimented with until the most satisfactory procedure for this investigation was developed.

Having an effective method of bond measurement, it was possible to determine relative mortar-aggregate bond strengths for a number of different aggregates. The aggregates investigated included a wide range of carbonate rocks indigeneous to the state of Indiana and other aggregates, both natural and artificial, which provided a desirable range of properties not available within the rocks of Indiana.

A number of aggregate characteristics and properties were evaluated as factors contributing to the bond between the aggregate and portland cement mortar. These included not only the aggregate type but also the grain structure, physical properties, and chemical composition.

The influence of bond strength on the strength of concrete was evaluated by flexural, indirect tensile, and compression tests of

concrete specimens. These specimens were prepared from aggregates for which mortar-aggregate bond strengths had previously been obtained.

A number of factors influence the strength of concrete, many of which are not directly related to the bond between the mortar and the aggregate. In order to obtain valid results comparing the effect of aggregates having various bond strengths, considerable effort was made to reduce the influence of the factors not directly related to bond. The same mortar was used throughout the investigation. The concrete was made of aggregate of only one size, thus eliminating the effect of gradation. All coarse aggregate was rounded to reduce the effect of particle shape and angularity.

Not all factors could be controlled, but every effort was made to enable the results of the concrete tests to most fully reflect the mortar-aggregate bond strengths.

EXPERIMENTAL DEVELOPMENT OF METHODS TO INVESTIGATE MORTAR-AGGREGATE BOND

Measurements of Bond

Methods of measuring bond used by previous investigators and described in the Introduction are physical tests from which result a measure of direct tensile strength or modulus of rupture. Other types of bond measurements were considered by the author in addition to methods used by previous investigators.

A characteristic of mortar-aggregate bond tests is that the inherent variability of the results is relatively large (1). Coefficients of variation of test results typically range from 10 to 30 percent. This large variation requires that a considerable number of tests be performed in order to produce meaningful results. As an example, for a probability of five parts in 100 that the difference between the sample estimate and the true value is greater than the allowable difference, a coefficient of variation of twenty percent, and an allowable sampling error of ten percent of the mean, sixteen test values must be obtained (13).

Methods previously used and reported, i.e., the use of a rock prism in a mortar briquet tested in direct tension and the use of a rock cube as the center portion of a prism tested in cantilever loading to measure the modulus of rupture, are both quite time consuming as they require individual casting of each test specimen. Nevertheless,

the author's first experimental work was with mortar-aggregate prisms based upon the method used by Alexander and Wardlaw (1, 9). They had obtained data on the mortar-aggregate bond and it seemed desirable to use the same technique.

Mortar-Aggregate Prisms

The author's mortar-aggregate prisms consisted of a small cube or rectangular prism of rock, one end of which had had mortar cast against it. The result was a rectangular prism one end of which was rock, the other of mortar.

The preparation of the aggregate into cubes of rectangular prisms, with a cross section about one inch square, was made even more laborious by the lack of a precise rock saw. A limited number of specimens were made having been cast in paraffin molds and stored in a fog room at 100 percent humidity. The specimens were removed from the molds in 24 hours and tested at the desired age with a falling lead shot cantilever loading device.

The bonded end of the prisms had been finished by grinding on glass plates with abrasives of various sizes or degrees of fineness so that the finish on the rock was either rough or very smooth. Mortar was used instead of cement paste because it was believed to more nearly represent conditions in concrete and would reduce possible shrinkage. The mortar initially used contained graded Ottawa sand. The water-cement ratio was 0.75 and the sand-cement ratio was 2.75, both by weight.

The results indicated that the bond for the smooth surface was greater than for the roughened surface. The conclusion was that the mortar was insufficiently "consolidated" against the vertical aggregate face. It was anticipated that this would continue to be a problem requiring extra attention in casting if it were to be overcome.

Mortar-Aggregate Cores

A new procedure for obtaining mortar-aggregate bond specimens was devised using a slab of rock, with one of its surfaces given a desired finish, on which was placed a plastic cement mortar. After a period of curing, this mortar-covered slab was cored with a diamond core drill and numerous individual specimens were obtained. The initial results were very encouraging and the procedure for preparing the specimens and for testing them was further developed and ultimately used for all mortar-aggregate bond strength tests in this investigation.

Preparation of Mortar-Rock Slabs. Rock slabs were cut with a concrete masonry saw from pieces of rock so as to expose the desired bedding planes of the rock. The dimensions of the slabs varied but a desirable size was found to be roughly 1 x 5 x 8 inches. This slab was then ground on an 18 inch diameter glass plate turning at 180 r.p.m. A succession of abrasive grits were used beginning with #100 aluminum oxide and water. This grit was used until the rock slab was uniform with all saw marks or other irregularities removed. To produce finely ground surfaces, which to the eye appear to have been polished, the slabs were subsequently finished with #240, #400, #800 and #1200

abrasive grits in that order, and all with water. Depending on the aggregate type, several hours were generally required to finish a finely ground surface. If a roughened surface was desired, the rock slab, after treatment with #100 aluminum oxide, was placed on a stationary glass plate covered with #60 silicon carbide abrasive grit and dry ground by hand only long enough to insure complete coverage. Care was taken to insure that the #60 grit did not break up to any great extent. It was replaced if the grinding required was more than five or ten hand revolutions. Maximum roughness with that specific size of grit was the desired result.

Initially, three different surface finishes - #60, #100, and #1200 abrasive grits - were considered. However, the extremes of #60 and #1200 were chosen for the bulk of the investigation.

The ground surfaces were cleaned with high pressure water and then stored until they were to be cast. Prior to casting if any contamination of the surface was believed to have been possible, the surface was rinsed several times with carbon tetrachloride. The slabs were then immersed in water to insure a saturated condition. Adjustable wooden molds wrapped in thin sheets of plastic were used to cast the slabs, Figure 1. Contamination of any surface with oil or other materials was, again, avoided. The slabs were cast in a saturated surface-dry condition obtained by drying the surface with a wet towel just before placing the cement mortar over the slab.

The mortar placed on the rock slab originally had the same proportions as that used in the rock-mortar prisms previously mentioned. Variations in the water-cement ratio were tried. On the basis of

observed workability and performance, a mortar mix with a water-cement ratio of 0.60 and a sand-cement ratio of 2.50 was selected for all further tests including all mortars reported in this investigation. A standard graded Ottawa sand and Type I portland cement were used. Details of these materials are given in the section of Materials.

Coring. The cast slabs were given the desired curing and then cored approximately seven days prior to testing. This meant that most of the coring was done at an age of seven days. The cores were cut seven days ahead of the testing date to accomodate equipment and labor scheduling.

A diamond core drill, Figure 2, 13/16 inch O.D., capable of holding a two inch specimen, was used in conjunction with a cooling and flushing liquid to obtain the final specimen from the aggregate mortar slab. The drill was mounted in a drill press and cooling liquid was supplied to the drill by either a hose connected to a water tap or by a recirculating pump, Figure 3. The latter was not used except for recirculating a water soluble cutting oil and water mixture to aid in coring the very hard aggregates. Some difficulty was experienced with the cutting oil when cutting mortar. The ground cement paste appeared to cause the oil emulsion to break and develop into a thick gummy material. The problem did not exist when cutting the hard aggregate, it was encountered only with the mortar. A trial with a neat cement paste presented such difficulties that it was necessary to discontinue the use of cutting oil with that material.

It was found that the best method of coring the slabs was by first coring the harder rock then passing through the mortar, Figure 4. The slab was underlaid with a thin piece of wood to prevent damaging the drill on the metal pan holding the slab.

After the cores had been cut from the slabs, they were generally left in their original position in the slab. This aided in selecting specimens to be tested for they were usually numbered prior to coring. It was found that considerable leaching of the mortar occurred if the cores were stored in a 100 percent humidity room. To avoid the leaching all specimens were placed in a saturated lime water solution after coring, in which they remained until time to be removed for testing.

Testing Mortar-Aggregate Cores. Three types of tests were tried on the cored specimens. The first was a cantilever type loading similar to that used on the prisms. The second was a direct shear test and the third was a torsion test.

The cantilever tests were performed with the aggregate portion of the specimens held in a vise arrangement and the mortar portion of the core loaded by a rolling frame. A photograph and detailed drawing of the apparatus are shown in Figures 5 and 7 respectively. This apparatus was placed on a direct shear soil testing machine, Figure 6, which provided the load. A double proving ring with an Ames dial allowed the load to be determined from the dial reading of inches x 10^{-4} inches per second. A reasonably uniform rate of loading was obtained by turning the handle by hand so as to have a change in the Ames dial reading of 10 x 10^{-4} inches per second.

Position of the bond plane with respect to line of application of the load was of utmost importance in testing. Care was taken to have the plane or interface between the mortar and the aggregate even with the top of the vise. If the interface was not exactly perpendicular to the axis of the core, the lowest point of the plane was positioned so as to be facing the load thus insuring that at the point of maximum tensile stress the vertical distance to the line of application of the load would always be constant. As shown in the detailed drawing of Figure 7 this distance was 0.523 inch. The apparatus detailed in Figure 7 worked well, however it would have been desirable to have the center of gravity of the loading frame ahead (moving in the direction of load) of the existing position for it tended to tip if light downward pressure were not kept on the forward end.

As may be noted by the semicircular hole in the rolling frame which was used to load the specimen, an attempt was made to grip the upper portion of the cylinder and then fail the specimen in nearly direct shear. Apparently the cores were sufficiently untrue to frequently cause the development of considerable bending stresses when the upper and the lower portions of the cores were independently gripped. Many failures occurred with no applied load and many more with a low load. Some, apparently gripped without bending stresses, developed a considerable load before breaking. The idea of testing in direct shear was abandoned due to the difficulty.

Considerable experimentation was done with a torsion test for the mortar-aggregate cores. The cores were cut with ends perpendicular to their axis. Square steel prisms were then glued to the end of the

cores with an epoxy cement. The specimen was placed in an apparatus which would keep the specimen aligned vertically and prevent bending. The bottom steel prism was held so it would not revolve while the upper prism could be twisted using a small torque wrench to measure the required force to cause failure.

This procedure appeared feasible until it became evident that the cores could not be allowed to dry, either prior to or during testing. This necessitated the glueing of the steel prisms to the cores under moist conditions. The available epoxies which performed well needed heat to insure curing in a reasonable time. In order to accomplish both the moist condition and the required heat for curing, a warm 130-140 degrees F. saturated limewater bath was used to cure the specimens after they had been bonded with an epoxy which would function under these conditions. The type of epoxy used to bond fresh concrete to old concrete was found to give moderate success. A successful bonding of about fifty percent of the cores was achieved and these were tested in torsion.

The torsion method was abandoned when the results from the cantilever tests indicated that it was detecting significant differences in the mortar-aggregate bond. Early and inconclusive torsion test results indicated that differences in the results would be within a narrow range thus making comparison difficult.

Several attempts were made to test cores made entirely of mortar. Apparently because these cores did not contain a plane of weakness, they did not produce meaningful data on the strength of mortar cores. Some of the fractures occurred partially within the portion gripped

in the vise while others broke well above the top of the vise. A disadvantage of this test is its inability to provide a value for cores made entirely of mortar.

Types of Mortar-Aggregate Core Failures. Some of the mortar-aggregate cores broke in such a way that the mortar-aggregate bond was not measured. Types of mortar-aggregate core failures are shown in Figure 10.

In the cantilever test the type of aggregate and the surface finish had an influence on the proportion of bond, partial bond, and non-bond failures. In most instances only the bond failures were considered as a valid result. The quartzite aggregates exhibited a tendency to fail with a small portion of mortar adhered to the leading edge of the rock core. Due to the difficulty of preparing the quartzite cores and with the predominance of this type of failure, these results were considered valid. If erroneous, they were lower than the true result would have been.

The torsion tests produced three types of bond failures. The clean bond failure was unusual. The cone and partial cone failures shown in Figure 10 were the more common. The latter two types indicate that under torsion the maximum shear stress was indeed along the perimeter of the bond plane in the mortar-aggregate core.

Effect of Drying the Cores. Early in the experimental development of the tests a number of cores were air dried for a period of approximately six weeks. At the end of this time the cores were tested and the resultant bond strengths were about one-third of the 14 day strengths obtained from a similar group of specimens that had not

been allowed to dry. This reduction in bond strength was similar to that observed by another investigator (5). Although the opposite of what occurs to the compressive strength of concrete which is dried, the reason for the loss was readily apparent from the broken ends of the cores. A roughly circular area toward the center of the core had a distinctly different appearance than the outer portion of the core. This was true of all dry cores and it was concluded that mortar shrinkage had caused the loss in strength and had indeed caused bond failure around the outer portion of the core prior to testing. The relative area over which bond failure due to shrinkage had occurred was probably a measure of the mortar-aggregate bond, but it was not sufficiently quantitative to pursue as a measure of bond strength. All further tests of mortar-aggregate bond strength were done in a saturated condition i.e., the specimens were saturated until immediately prior to testing.

Experimentation with various methods of preparing a relatively large number (twenty or more) of mortar-aggregate test specimens, and of testing these specimens to obtain a measure of mortar-aggregate bond strength, resulted in the selection of one method for the remainder of the investigation. This method was the preparation of mortar-aggregate cores which were kept moist at all times until tested in a cantilever loading.

PLAN OF THE INVESTIGATION

The investigation consisted of two separate phases. The first phase dealt with measurements of the bond between portland cement mortar and coarse aggregates. The second phase of the investigation dealt with concrete made with some of the aggregates whose mortar-aggregate bond strength had been determined in the first part. In this investigation, bond is defined as the effective sum of both mechanical and chemical bonding as measured by the cantilever loading of mortar-aggregate bond test specimens.

In the first phase, insofar as possible, the only variables were those of the aggregates themselves. The variables included the type of rock (chemical and minerological composition), surface finish, grain size, lithic properties, and physical properties such as modulus of elasticity, specific gravity, absorption and particle shape.

The aggregates used were generally those of acceptable quality for concrete and the bulk of them were limestones and dolomites from the state of Indiana. Some other materials were used in order to obtain a spread in the bond strength and in order to obtain control over some of the variables.

The test specimens for each aggregate were prepared from a single piece of rock. This rock was cut into slabs and the slabs given the desired surface treatment. One side of the finished slab was covered with portland cement mortar and allowed to cure. The individual

test specimens were cut from the mortar covered slab with a core drill. These specimens were then tested in a cantilever loading.

In order to minimize the variables in the second phase of the investigation, which dealt with concrete made with some of the aggregates whose mortar-aggregate bond strength had been determined in the first part, one-size coarse aggregate was used with the portland cement mortar to make the concrete. The coarse aggregate was selected at the quarry to minimize variations among various pieces of any given aggregate. The crushing was done in the laboratory and the crushed rock was rounded in order to obtain a more uniform surface finish and shape among the different aggregates.

It was desirable to have some type of "aggregate" with a very low mortar bond strength, which could be used in a concrete, to obtain a complete range in mortar-aggregate bond strengths. Two types of plastic, polystyrene and plexiglas were readily available for experimentation. It was not possible to core these plastics; however, mortar was cast against finished surfaces of the material and it was found that essentially no bond was developed on sheets of plexiglas, while some was developed on the polystyrene. Plexiglas was therefore selected as a concrete "aggregate". The individual pieces were obtained by cutting up plastic rods.

Glass marbles provided a material with a bond strength midway between the very low strength obtained with plexiglas and the relatively high strengths of the mineral aggregates.

The same mortar was used in the concrete as had been used in the bond tests. As much of one-size aggregate as possible was worked

into the concrete specimen molds. This was to eliminate segregation and to insure a near constant mortar-aggregate ratio.

Tests run on the concrete included both flexural and indirect tensile tests of beams. Compression tests were run on concrete cylinders in which part of the tests included measurements of the occurrence of cracking impulses believed to be caused by microcracking of the concrete.

The small vibratory impulses created by the microcracks were detected by utilizing an arrangement with a phonograph cartridge. The electrical impulse received from the cartridge was amplified and viewed by means of an oscilloscope. The impulses were counted with a high speed counter analyzer. The results of the impulse counts gave some insight into the effect of bond strength on microcracking in a concrete subjected to compressive loads.

MATERIALS

Coarse Aggregates

The natural aggregates used in the investigation were chosen so as to represent a wide range of properties, but with nearly all being acceptable aggregate for concrete. The pieces of rock were carefully selected from shot piles at the various quarries. Care was taken to insure that all pieces representing an aggregate came from the same strata. In one instance a slight variation in color indicated that a dissimilar rock had been obtained and crushed for concrete. A petrographic thin section was made of the rock in question in order to determine if it were significantly different. The examination showed no differences in the rocks.

The properties of the aggregates used are listed in Tables 1 and 2, Aggregate Properties. The numbers were assigned to the various aggregates for the purpose of identification in this investigation.

The name of the rock identifies the geologic formation from which it was obtained and the general rock type. Considerable variation in the rock occurs within many of the formations, thus several aggregates with different properties may come from the same formation.

The properties of the aggregates, given in Tables 1 and 2, include petrographic analysis, chemical analysis, and physical properties. The physical properties include the modulus of elasticity determined from stress-strain data obtained by loading cores of the aggregate in

TABLE 1

PETROGRAPHIC AND CHEMICAL PROPERTIES OF AGGREGATES

	Aggregate	Petrographic Description		Chemical Analysis* Percent by Weight		
No.	Name	Average Grain Size, millimeters	Lithic Properties	Dolomite	Calcite	Insoluble Residue
2	Jeffersonville (Dolomite)	0.005	Carbonate, very fine grained, crystalline	98	1	1
6	Liston Creek (Dolomite)	0.02	Carbonate, very porous, rombic	97		3
7	Bariboo (Quartzite)	0.16	Quartz, interlocking structure			
8	Bariboo (Quartzite)	0.25	Quartz, interlocking structure, "veins" of microquartz			
9	St. Genevieve (Limestone)	0.01	Carbonate, fossiliferous detrital quartz, opaque metallics	10	88	2
10	St. Genevieve (Limestone)	0.08	Carbonate, fossiliferous, opaque metallics	2	92	5
11	Lincoln "Quartzite" (Sandstone)	0.17	Angular quartz grains in a carbonate matrix			
12	(Diorite)					
13	Harrodsburg (Limestone)	0.40	Carbonate, fossiliferous, detrital quartz and opaque metallics	6	95	1
14	Geneva (Dolomite)	0.04	Carbonate, very porous, rombic	99		1
15	Louisville (Dolomitic Limestone)	0.03	Carbonate, some large fossils, rombic crystals, opaque metallics	60	37	3

* Calculated from laboratory determinations of calcium, magnesium and insoluble residue contents.

TABLE 2

PHYSICAL PROPERTIES OF AGGREGATES

		Rounded and Washed, Passing 1 inch Retained 3/4 inch				
No.	Name	Modulus of Elasticity, $Ex10^6$	Bulk Spec. Gravity	Absorption percent	Particle Shape $L/_W$ *	$W/_T$ *
2	Jeffersonville (Dolomite)		2.67	1.29	1.33	1.81
6	Liston Creek (Dolomite)		2.58	1.78	1.41	1.52
7	Bariboo (Quartzite)	13.8		0.20	1.42	1.43
8	Bariboo (Quartzite)					
9	St. Genevieve (Limestone)	11.0	2.69	0.42	1.39	1.74
10	St. Genevieve (Limestone)	12.1	2.68	0.46	1.34	1.53
11	Lincoln "Quartzite" (Sandstone)					
12	(Diorite)					
13	Harrodsburg (Limestone)	10.1	2.65	0.66	1.36	1.69
14	Geneva (Dolomite)	10.4	2.43	3.43	1.37	1.55
15	Louisville (Dolomitic Limestone)	15.1	2.76	0.62	1.38	1.59
16	Gravel (Quartzite)		2.62	0.22	1.30	4.42

*L, The mean greatest dimension of pieces of aggregate

W, The mean intermediate dimension of pieces of aggregate

T, The mean smallest dimension of pieces of aggregate

compression, (see the Appendix for a detailed description). Other physical properties, specific gravity, absorption and particle shape were determined on the aggregates after they had been crushed, rounded and separated into a one-size fraction passing a one inch sieve and retained on a three-fourths inch sieve.

Several aggregates warrant special comment in addition to their properties listed in Table 1. Aggregate No. 6, Liston Creek dolomite, is macro-porous and approaches what has been called reef rock. It meets the normal acceptance tests for a high quality concrete aggregate. Aggregate No. 14, Geneva dolomite, is also macro-porous but with an even higher absorption value. It does not meet the normal acceptance tests for a high quality concrete aggregate. In comparing the two aggregates, No. 6 and No. 14, the former was more uniformly porous than the latter whose more porous portions were avoided when bond tests were made.

Three of the aggregates, No. 9 and No. 10, both St. Genevieve limestone, and No. 13, Harrodsburg limestone, had similar properties except for their grain size and fossil size. The fossil size seemed to be in proportion to the size of the crystal grains. These three aggregates were used to investigate the effect of grain size.

When the pieces of aggregate were being selected from the shot piles, an attempt was always made to select one or more pieces which could readily be cut up into slabs for the mortar-bond test. The preparation of these slabs will be given in detail under the procedure for the bond test.

Aggregate pieces which were not used in the bond test were broken initially with a sledge hammer until they would fit a four-inch jaw crusher. After crushing, the rock was separated on a three-fourths inch sieve. That retained was ready to be rounded.

Rounding of the aggregate was done to minimize the difference in both angularity and particle shape among the aggregates. This was done by placing the rock in a Los Angeles abrasion machine along with some coarse abrasive grit and enough water to cover the aggregate. The lid to the Los Angeles machine was sealed with a plastic non-hardening gasket compound to prevent leakage. The machine was operated long enough to give a rounded aggregate without excessive reduction in the aggregate size. The time required varied with the aggregate, and the amount of abrasive, but ranged from three to ten hours.

The rounded aggregate and the slurry were removed from the Los Angeles machine with a large vacuum cleaner thus eliminating many of the clean up difficulties with equipment and facilities. The aggregate was washed, dried, and again sieved so that only rounded and washed aggregate passing the one inch sieve and retained on the three-fourths inch sieve was used in the preparation of concrete.

Mortar Sand

The sand used in all of the mortar including that in the concrete was natural silica sand from Ottawa, Illinois, generally referred to as graded Ottawa sand. It conformed with the requirements of ASTM Designation: C 109, Compressive Strength of Hydraulic Cement Mortars.

Cement

A Type I portland cement, conforming to ASTM Designation: C 150-64, Standard Specifications for Portland Cement, and all from the same clinker batch was used throughout the investigation. The laboratory designation of this cement was No. 316. The physical and chemical properties of the cement are given in Table 3.

A concrete was made, with the cement, having a water-cement ratio of 0.60 and a cement factor of 5 1/4 sacks per cubic yard. The average 7, 14, and 28 day compressive strengths were as follows:

7 day compressive strength	3840 psi
14 day compressive strength	4620 psi
28 day compressive strength	5260 psi

Mortar

Mortar of the same proportions was used throughout the investigation. The water-cement ratio for the mortar was 0.60 and the ratio of the graded Ottawa sand to cement was 2.50. Whenever feasible, the mix was of a size that the water content equalled the capacity of a standard calibrated flask. The graded Ottawa sand was weighed to the nearest gram.

TABLE 3

PHYSICAL AND CHEMICAL PROPERTIES OF CEMENT 316

Physical Properties

Fineness, No. 325 sieve	93.6 percent
Specific Surface, Blaine	3320 sq. cm. per gm.
Initial set	3 hr. 20 min.
Final set	5 hr. 20 min.
Air Entrained (ASTM Designation: C 185-59)	6.6 percent

Chemical Analysis

Compound	Percentage present
Silicon dioxide, SiO_2	22.01
Aluminum oxide, Al_2O_3	5.38
Ferric oxide, Fe_2O_3	2.15
Calcium oxide, CaO	65.40
Magnesium oxide, MgO	0.75
Sulphur trioxide, SO_3	2.47
Loss on ignition	1.25
Total Alkalies	0.30

Calculated Compound Composition

Compound	Percentage present
Tricalcium Silicate, C_3S	49.45
Dicalcium Silicate, C_2S	25.87
Tricalcium Aluminate, C_3A	10.60
Tetracalcium Aluminoferrite, C_4AF	6.54
Calcium Sulphate, $CaSO_4$	4.20

PROCEDURES

The details of the methods used in conducting the mortar-aggregate bond tests and the tests on concrete are given in this section. This includes the details of measurements of the number of microcracks, by impulse counts, which occur in the compression tests of concrete cylinders.

Mortar-Aggregate Bond Tests

The details of all the procedures, from the preparation of the slabs to the actual testing of the cored specimens, are described in this section on mortar-aggregate bond tests.

Preparation of Slabs

The rock slabs used in the mortar-aggregate bond tests were generally prepared several days prior to having the mortar cast on them. A piece of rock of a convenient size for cutting on a masonry saw was selected. The most convenient size was that which would give a slab with a cross-section approximately five by eight inches. Many times irregular shaped slabs were obtained which were satisfactory so long as they could be properly finished. The thickness of the slabs was as close to one inch as could easily be cut on the masonry saw.

The slabs were cut perpendicular to any bedding planes which existed. This was to insure that a representative value for an aggre-

gate would be obtained. Had cuts parallel to the bedding been used, the bond might have been determined for a thin stratum of material not representative of the aggregate.

Irregular pieces of rock were cast in a shallow base of concrete mortar so that they could be firmly held during cutting.

A water cooled wet cutting, eighteen inches diameter, diamond blade of the type recommended for concrete was used to cut the slices from the rock. The pieces were fed into the saw by hand. In general, a number of parallel slabs were cut from a given piece of rock and these slabs were used for all mortar-aggregate bond tests representing that aggregate source.

After the slabs had been cut, one side was selected for finishing. Two surface finishes were prepared on each aggregate, one slab smooth with a polished appearance, and another slab rough. For the smooth slabs, one side was initially finished with No. 100 aluminum oxide abrasive grit and water on a glass plate revolving at 180 rpm. The plate wore with the grinding making frequent replacement necessary. When a group of slabs had been finished with the No. 100 abrasive grit so that all saw marks and other irregularities had been removed, progressively finer grits were used along with water to give the smooth finish. The abrasive grits were No. 240, No. 600, No. 800 and No. 1200, all aluminum oxide. The length of time required to grind the slab with each size of abrasive grit varied not only with the rock but also with the pressure placed on the rock by the person doing the polishing. Frequently, weights were placed on the rock to aid the grinding. As a general rule thirty minutes with each grit, except the No. 1200 grit

which had two thirty minute periods, was sufficient for the grinding process. The hard quartzites required much longer periods of grinding. Considerable care was taken to insure that the glass plate and the aggregate slabs were well cleaned before a finer abrasive grit was used.

Those slabs given a rough finish were first finished with the No. 100 abrasive grit, then dried, and finally finished on a stationary glass plate with dry No. 60 silicon carbide abrasive grit. Care was taken to insure that the grit did not break up excessively and it was replaced as necessary. When dealing with the quartzites, a large slab of quartzite was substituted for the glass plate, owing to the quartzite being so much harder than the glass.

All slabs were thoroughly flushed with a high pressure water spray after being finished. In the case of the wet finishing it was important to do this before the surface dried.

The finished surfaces were protected from scratches with paper and then stored until they were to be used. Before using, the surfaces were washed with carbon tetrachloride whenever there was any possibility of their having been contaminated with oil in handling or by other means. All slabs were soaked in water prior to casting to insure a saturated condition.

Casting Mortar-Aggregate Slabs

The mortar was mixed in a small mechanical mixer with one-half minute of dry mixing and two and one-half minutes of wet mixing. A

water cement ratio of 0.60 and a sand cement ratio of 2.50 was used. The sand and cement were weighed out on a 4500 gram capacity torsion balance, and the water was measured volumetrically, generally in a calibrated flask. The usual mix was as follows:

Cement, Type I, Laboratory Designation No. 316	833 gm.
Graded Ottawa Sand	2,032 gm.
Water	500 cc.

The molds used for casting the mortar on the aggregate slabs were made of wood wrapped in plastic, Figure 1. The clean but saturated aggregate slab was placed in the mold with the finished side up. A clean moist towel was placed over the rock to insure that it would not dry. The towel was removed along with any free water on the slab immediately before the mortar was poured onto it. The mortar was worked with a large putty knife to insure complete contact with the slab surface and to work out any bubbles. A one inch depth of mortar over the aggregate slab was required. In order to have this it was occasionally necessary to use two batches of mortar. When sufficient mortar was in place, the form was vibrated by hand to work out entrapped air bubbles. This was important in preventing bubbles from being next to the aggregate surface. The top of the mortar was then leveled and allowed to set. The mold was covered with a sheet of plastic which in turn was covered with wet cloths. The cast mortar-aggregate slab was removed from the mold in about twenty-four hours and placed in a fog room for curing.

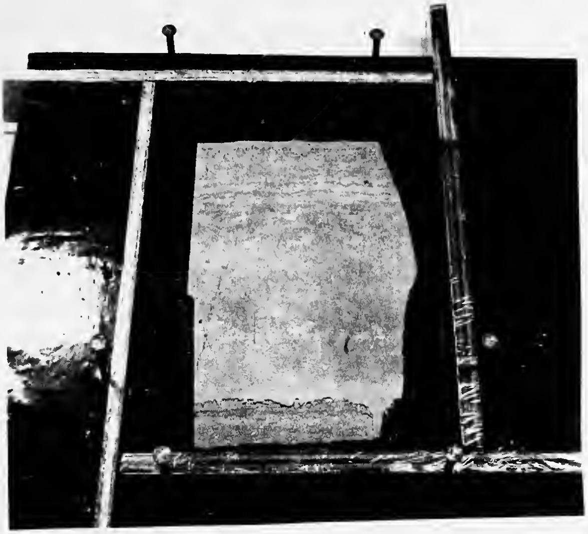


FIGURE I. MOLD AND FINISHED ROCK SLAB PRIOR TO CASTING

FIGURE 2. DIAMOND CORE DRILL

Coring Mortar-Aggregate Slabs

The mortar-aggregate slabs remained in the fog room until they were cored. The coring was done approximately seven days prior to testing. A diamond core drill, Figure 2, 13/16 inch O.D., capable of holding a two inch core was used to make the cores. The drill was mounted in a drill press and supplied with either water or a water and cutting oil mixture, Figure 3. The water soluble cutting oil was necessary to core the quartzite aggregates. The mortar-aggregate slabs were placed rock-side up in a pan for coring, with a thin piece of wood beneath the mortar to protect the drill, Figure 4.

After the cores had been made they were generally left in their original location in the slab and the slab, with the cores, was then placed in a saturated lime water solution until they were to be tested.

Testing the Cores

The apparatus used to test the mortar-aggregate cores is shown in Figures 5, 6 and 7. Figures 5 and 7, respectively, show a photograph and a detail drawing of the apparatus. Figure 6 is a photograph of the apparatus in the direct shear soil testing machine used to provide and measure the load.

The cores were tested at the desired age which was generally fourteen days. They were kept in the saturated lime water until they were placed in the testing apparatus. The rock portion of the core was placed down into the vise arrangement and the core positioned so that the interface between the mortar and the rock was even with the top of

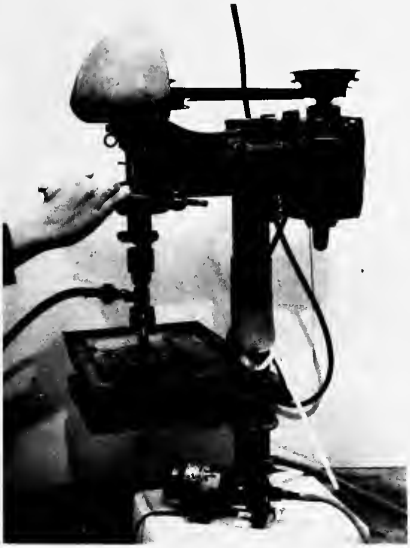


FIGURE 3. CORING EQUIPMENT WITH RECIRCULATING PUMP FOR COOLING LIQUID

FIGURE 4. A PARTIALLY CORED SLAB

the vise. If the interface was not exactly parallel to the vise, the lowest portion was turned so as to be nearest to the load, to the readers right as he looks at Figure 6, and even with the top of the vise.

The load was applied through the double proving ring. A consistent rate of loading was obtained by turning the handle so that the Ames dial showed a change in reading of 10×10^{-4} inch per second. This is equivalent to three pounds per second for loads recorded in the investigation.

The dial reading at failure was recorded to the nearest 5×10^{-4} inches shown on the Ames dial for each core, along with any pertinent information on the type of failure. Non-bond failures, such as failures through the rock or mortar, were not used in the analysis of the data. The dial reading was converted to the load in pounds and it was this which was used for analysis. The mean, $\bar{x}$, standard deviation, σ, and coefficient of variance, v, were determined for the data observed on all cores breaking in bond from each mortar-aggregate slab.

Tests on Concrete Made with Selected Aggregates

The procedures followed in preparing and testing the concrete made to determine the influence of bond strength on the strength of concrete are described in this section. Flexural and indirect tensile tests were performed on concrete beams and compression tests were conducted on concrete cylinders.

Casting of the Specimens

All concrete was made from one-size coarse aggregate. The mortar

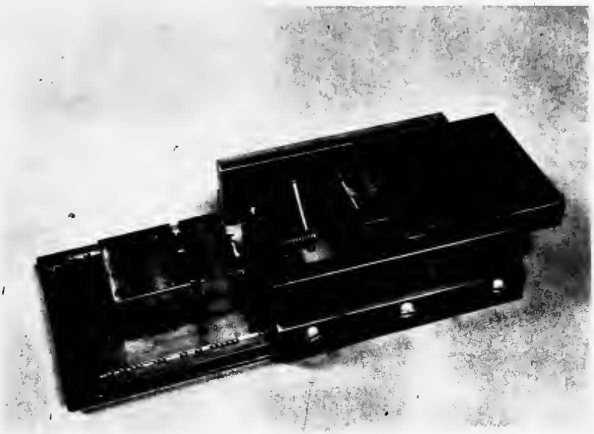
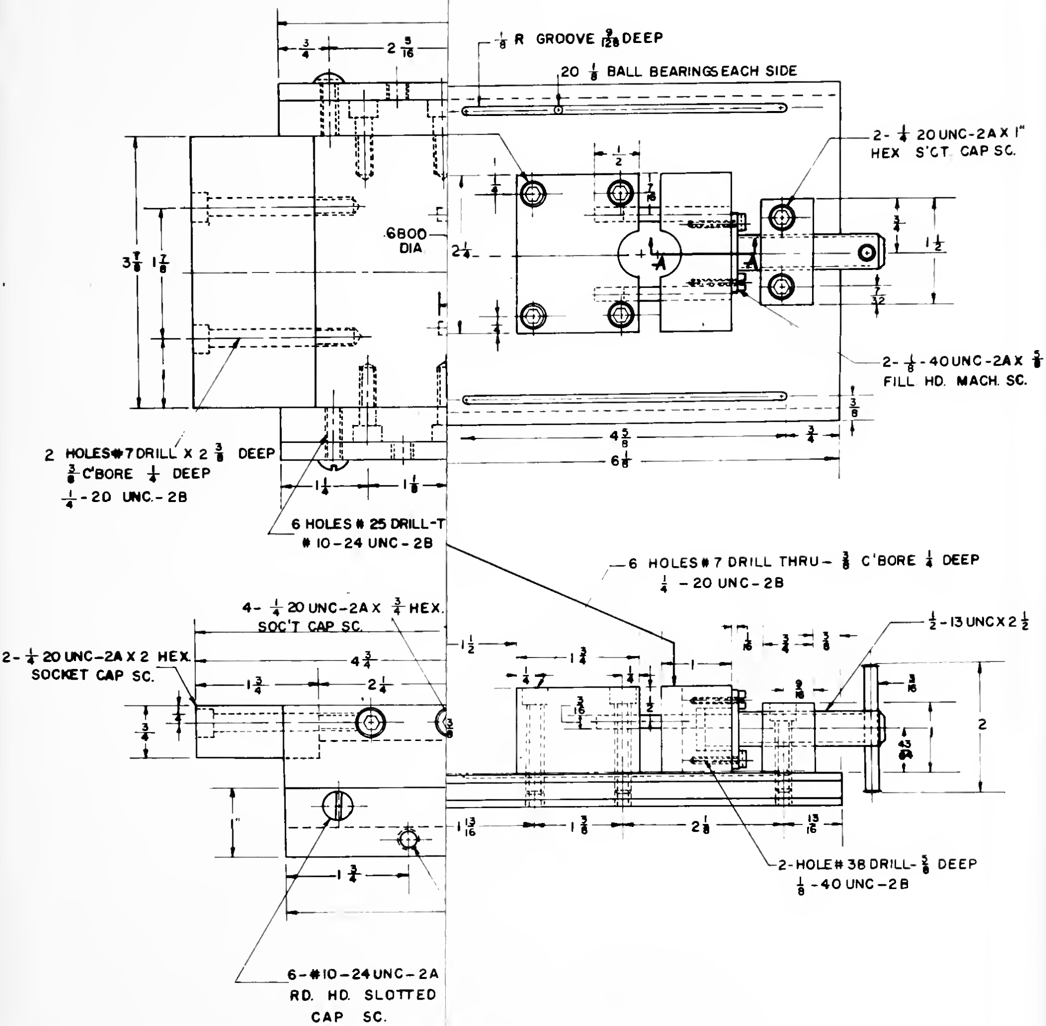


FIGURE 5. LOADING APPARATUS FOR MORTAR-AGGREGATE CORES

FIGURE 6. MORTAR-AGGREGATE CORE IN PLACE BEFORE TESTING

)TTOM ASSEMBLY



FIG(GREGATE CORES

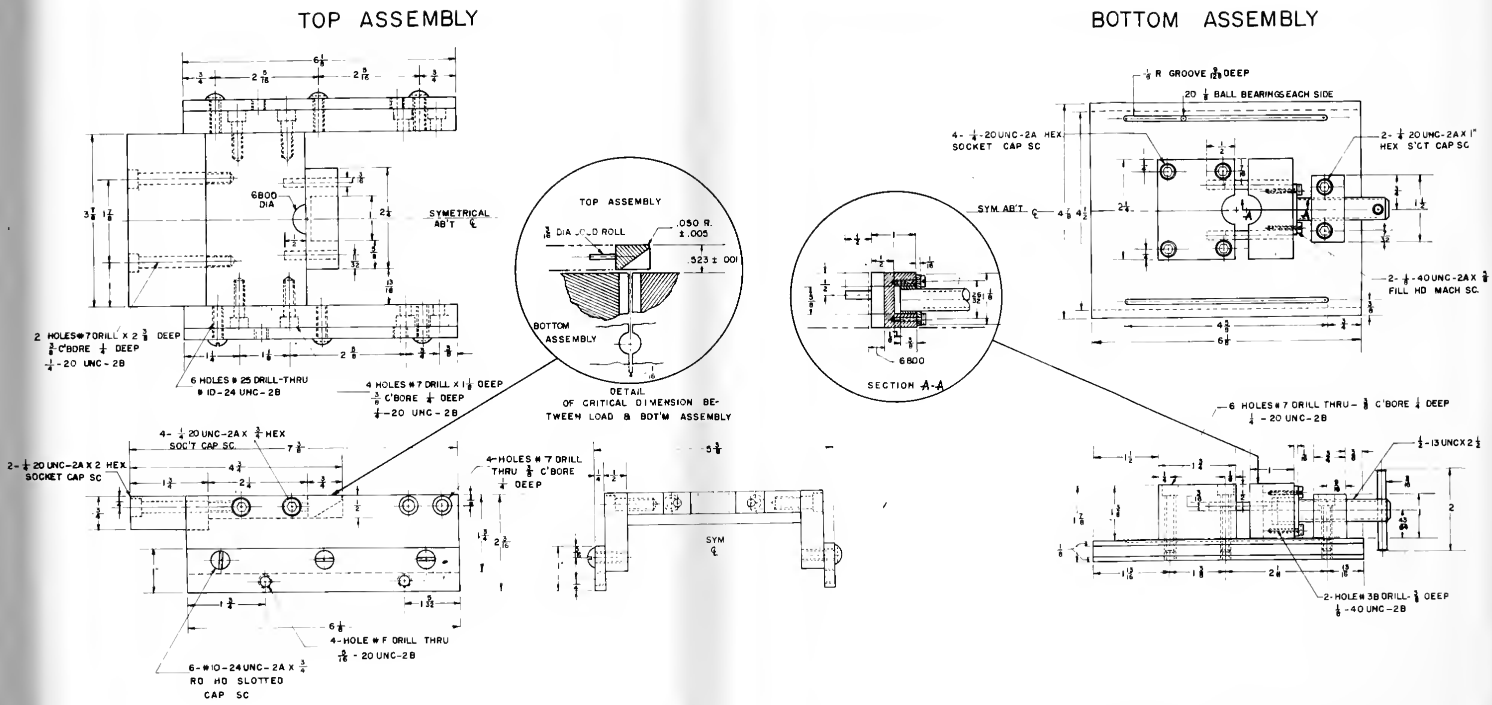


FIGURE 7. DETAIL DRAWING OF LOADING APPARATUS FOR MORTAR-AGGREGATE CORES

fraction was the graded Ottawa sand mortar used throughout the previously described phase of the investigation. The same materials used in the mortar-bond test were also used in the concrete tests. The aggregate was selected to represent the same rock which had been tested in mortar-aggregate bond. It was rounded, washed, and in a saturated surface dry condition when it was placed with the same mortar used in the mortar-bond tests.

The mix used for the concretes had a water cement ratio of 0.60 and a sand cement ratio of 2.50 by weight. The cement, sand, coarse aggregate and water were mixed as for a normal concrete. The mold was filled in a normal manner, except that as much coarse aggregate as possible was worked into the mold in order to prevent segregation of the coarse aggregate in the mix. This necessitated that it be rodded a large number of times, the actual number varying with the aggregate. The resulting concrete had no segregation and it also had quite a uniform volume of coarse aggregate in each specimen, Table 8, regardless of the aggregate. The volume of coarse aggregate in the concrete was close to sixty percent in all cases. No air entraining agent was used in the concrete.

The mixing was done in a small mechanical mixer except for the concrete wherein the coarse aggregate was rock cores left from the bond tests. These jammed the mixer so often that they were mixed by hand. The sand and cement were mixed together before adding the coarse aggregate and the water. A total time of not less than two minutes of wet mixing was used on the concrete.

Materials for concrete specimens, either 2 x 2 x 11 1/4 inch beams

or 3 x 6 inch cylinders, were individually batched and mixed. The specimens were cast by rodding and working as much coarse aggregate into them as was possible. This not only prevented segregation of the one size mix but also resulted in a rather constant proportion of coarse aggregate in the mixes. The coarse aggregate from a mix that was not used was washed, dried and weighed so that the actual dry weight of coarse aggregate in each specimen was determined. From this the volume of coarse aggregate in each specimen was calculated using the bulk specific gravity for the coarse aggregate.

Many of the concrete cylindershad a thin hooked wire cast into their side about one inch from the top. This was later used in counting the cracking impulses.

After casting, the concrete was allowed to set and then covered with plastic and wet cloths. The specimens were removed from the molds in about twenty-four hours and placed in a fog room until they were to be tested. The testing was done at an age of fourteen days in this investigation.

Testing Concrete Beams

Each concrete beam was marked with pencil to indicate the locations to be broken in the indirect tensile splitting test. The dimensions of the beam at each of these marks was measured and recorded. The beam was first broken in flexure in accordance with ASTM Designation: C 293, Flexural Strength of Concrete (Using Simple Beam with Center-point Loading). The modulus of rupture was calculated from the results.

Two indirect tensile tests were performed on each of the halves of

the beams. Two 5/32 inch steel rods were used to apply the load to the beam for the test. A flexible foam plastic sheet lightly supported the free ends of the beam as it was loaded at the rate of 1000 pounds per minute. The "tensile" stress was calculated using the following formula:

$$\text{Tensile Stress, } S_T = \frac{KP}{bd} \tag{16}$$

when:

K = Constant for a rectangular beam = 0.648

P = Load

b = Width of beam at point of failure

d = Depth of beam at point of failure

Testing Concrete Cylinders

The concrete cylinders, having been capped with a sulfur compound, were tested in compression according to ASTM Designation: C 39, Compressive Strength of Molded Concrete Cylinders, except for the rate of loading. A rate of 8,000 pounds per minute was used and resulted in a stress increase slightly lower than the minimum of 20 psi per second given in the ASTM method. The lower rate was used in order to allow time for the reading of the impulse counts.

Measurements of Microcracking by Impulse Counts. The influence that the mortar-aggregate bond apparently has on microcracking is such that it appeared desirable to have some means of measuring microcracking in the concrete specimens. It was anticipated that the onset of severe microcracking might be an indirect measure of bond strength.

Two methods have been used to indirectly measure the formation of microcracks. The method used by Jones and Kaplan (4) was an ultrasonic

pulse technique. Another method is to listen to the sound of the cracking using a microphone and high amplification such as reported by L'Hermite (14). Of these methods, the latter was used in this investigation to develop a method of measuring microcracks occurring in compression tests.

Initially a vibratory type microphone was attached to the side of the concrete cylinder with plastic tape, and the amplified sound was recorded with a tape recorder. The results were interesting, but insufficient and lacking in any quantitative measure. The microphone was apparently only sensitive enough to pick up the loudest cracking noises.

Two refinements in technique were then tried, both of which proved successful. To replace the microphone with a more sensitive apparatus, a small stiff wire was embedded into the side of the cylinder, Figure 8, and a phonograph cartridge placed on the end of the wire. The wire was essentially acting as a phonograph needle. This arrangement proved to be far more sensitive than the microphone, although it did require that a piece of wire be placed in the side of the cylinder when it was cast. A small bend or hook on the wire, within the cylinder, prevented the wire from breaking its bond within the cylinder.

The other refinement was to replace or supplement the tape recorded sound with an oscilloscope on which it was possible to see the vibratory impulses of the cracking. This impulse method proved to be much better than the sonic method. It was possible to obtain an estimate of the frequency and the amplitude of the vibratory impulses picked up by the microphone. Interference noises could easily be identified on the

FIGURE 8. BROKEN COMPRESSION TEST CYLINDER WITH EMBEDDED WIRE

oscilloscope. In order to obtain a quantitative measure of the impulses, a counter analyzer was added to the apparatus to record the number of impulses picked up. It was necessary to adjust the counter prior to each testing period; therefore the sensitivity was not always the same, and the number of counts recorded was not necessarily an indication of the actual number of crack vibrations. However, it did definitely indicate the initiation of cracking and the relative rate of cracking among tests performed at a given test period.

Procedure for Obtaining the Impulse Counts

The equipment used to obtain the impulse counts from the 3 x 6 inch cylinder cast with a wire extending from the side, Figure 8, is listed below.

Phonograph Pickup, Astatic, Model 12u.

Preamplifier, Transistorized 2 stage, Frequency response Flat, 50 cps - 100 kcps.

Oscilloscope, Tektronix Inc., Model 543A with type C-A Plug-in.

Counter Analyzer, Computer Measurements Co., Model 225B.

The cylinders were cast, cured, and capped following the procedure used on other cylinders and beams. At the desired age they were placed in a hydraulic testing machine, instead of a mechanical machine, to minimize vibrations. The cylinders were then tested in accordance with ASTM Designation: C 39 - 56T, Compressive Strength of Concrete Cylinders, except that the rate of loading was 8,000 pounds per minute in order to allow time to read the counts at load intervals of 1000 pounds.

The cumulative impulse count was read and recorded along with the

load. The maximum load was recorded; however, an impulse count was not generally obtained unless failure coincided with a 1000 pound interval of load.

Some initial impulses were obtained under very low loads and were believed attributable to adjustment of the cylinder cap. In all instances the cracking nearly ceased at a low load and did not commence again until thirty to seventy percent of the ultimate load had been reached.

The cumulative count versus the load in pounds was plotted. Figure 9 is an example of such curves. The inflection load, or that load at which a significant increase in rate of impulses occurred, was determined by extending the two essentially straight line portions of the curves until they intersected. This was arbitrarily taken as the inflection load. The inflection stress, in pounds per square inch or psi, was obtained by dividing the inflection load, in pounds, by the area, in inches.

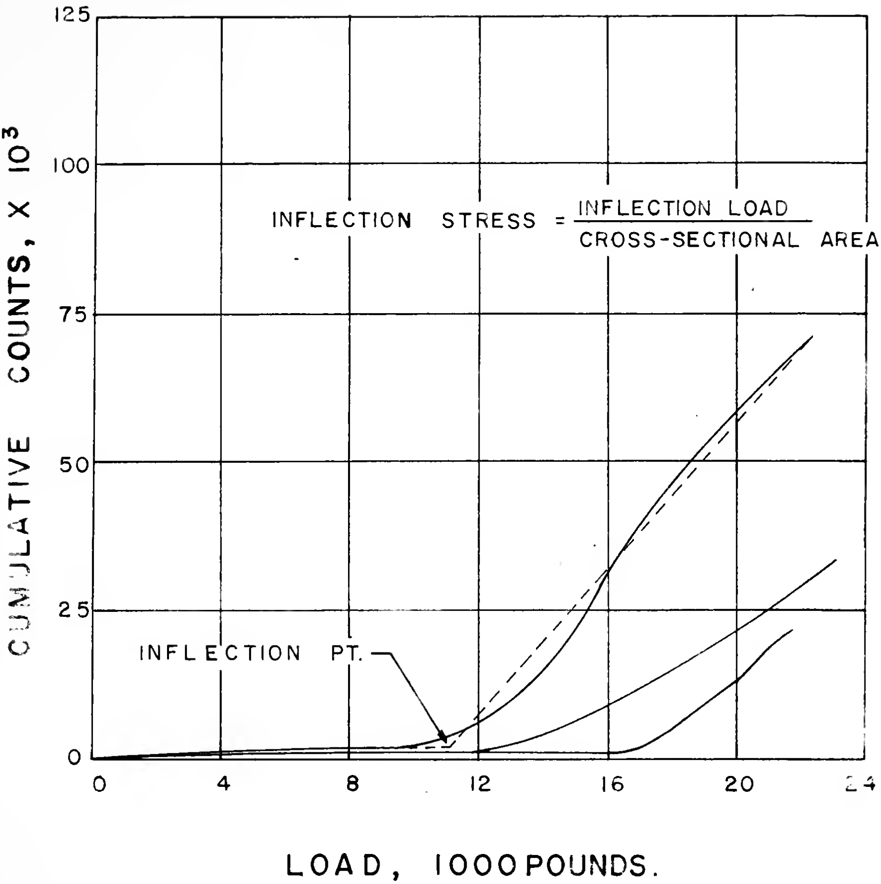


FIGURE 9. CUMULATIVE IMPLUSE COUNTS VERSUS LOAD IN THE COMPRESSION OF CONCRETE CYLINDERS

RESULTS OF MORTAR-AGGREGATE BOND TESTS

The mortar-aggregate bond tests, conducted by breaking core specimens in cantilever loading, provide information by which some of the factors that may influence the bond between mortar and aggregate can be examined. These factors include the effects of: 1. aggregate type, 2. surface finish, 3. age when tested, and 4. grain size of the aggregate. Some insight into the relative amounts of chemical and mechanical bonding may be obtained from the results.

The results of the bond tests at 14 days are summarized in Table 4. Photographs of various types of failures are shown in Figure 10. Only bond-type failures were used as acceptable data. To aid in determining whether differences in results were significant, three separate analyses of variance (ANOV) were made. These analyses considered the variables of surface finish and aggregate type for three selected groups of aggregate types. The ANOV plans, tables and analyses are in the Appendix.

The means of each ANOV were analysed by the Neuman-Kuels technique in order to determine which means were significantly different.

Effect of the Kind of Aggregate

All three of the ANOV's showed that a significant difference in the mortar-aggregate bond strength was obtained for different

TABLE 4

SUMMARY OF RESULTS OF MORTAR-AGGREGATE BOND TESTS

14 days, w/c = 0.60, s/c = 2.50

Aggregate No.	Aggregate Name	Finished No. 60 Grit $\bar{\bar{x}}$ lbs.	V	n_t	Finished No. 1200 Grit $\bar{\bar{x}}$ lbs.	V	n_t
2	Jeffersonville (Dolomite)	61	19	19	87	10	46
6	Liston Creek (Dolomite)	65	17	27	53	15	26
7	Bariboo (Quartzite)	80	8	32	84	10	22
8	Bariboo (Quartzite)	72	12	27	73	9	11
9	St. Genevieve (Limestone)	63	12	78	58	26	33
10	St. Genevieve (Limestone)	50	18	56	51	25	39
11	Lincoln Quartzite (Sandstone)	76	11	46	56	18	14
12	Diorite	53	16	20	56	12	22
13	Harrodsburg (Limestone)	64	13	83	54	25	35
14	Geneva (Dolomite)	58	14	65	69	18	29
15	Louisville (Dolomitic Limestone)	57	18	21	74	10	30
	Neat Cement	32	21	31	48	27	33
	Plate Glass	45	10	52	32	13	28

$\bar{\bar{x}}$, Overall mean value of the load to cause bond failure, in pounds.

V, Coefficient of Variation for the cores successfully tested, in percent.

n_t, Number of cores

w/c, Water-cement ratio

s/c, Sand-cement ratio

A

CONE FAILURE
(TORSION TEST)

B

PARTIAL CONE FAILURE
(TORSION TEST)

C

BOND FAILURE
(CANTILEVER TEST)

D

MORTAR FAILURE
(CANTILEVER TEST)

E

PARTIAL MORTAR FAILURE
(CANTILEVER TEST)

FIGURE 10. TYPES OF MORTAR AGGREGATE CORE FAILURES

aggregates. This is most clearly shown in the Neuman-Kuels analyses of ANOV's 2 and 3. In ANOV 2, the mean for aggregate No. 10 was significantly lower than the means for both aggregates No. 9 and No. 13. In ANOV 3, the two aggregates were both quartzite, but the mean bond strength of one was significantly lower than the others. Both quartzites had high mean bond strengths compared to the carbonate aggregates, regardless of the surface finish. It is believed that this is due primarily to chemical bonding. As a general statement, the analyses permit one to say that a difference between means of 10 pounds or more is significant for an α of 0.05.

The summary of mortar-aggregate bond test results in Table 4 shows that the bond strength as indicated by the mean load, $\bar{\bar{X}}$, varies between 50 and 90 pounds for the mineral aggregates tested. The artificial "aggregates", one composed of neat cement and the other of plate glass, both produced bond strengths lower than obtained with the mineral aggregates. The kind of aggregate does influence the bond strength.

Effect of Surface Finish

The influence of the surface finish is not consistent with different carbonate rocks. ANOV's 2 and 3 both showed that the surface finish was not a significant factor. Neither was there a significant interaction between the aggregate and the surface finish, thus the finish had no significant effect. ANOV 1 showed a significant interaction between the aggregate and the surface finish although surface finish itself was again not significant. The Neuman-Kuels analysis of ANOV 1 shows the interaction occurring because aggregates

4 and 6 were both influenced by the surface treatment, but in opposite ways. Aggregate 4 had a greater bond with the No. 60 grit than with the No. 1200 grit, but aggregate 6 had less bond with the No. 60 grit than with the 1200 grit. The third aggregate in the analysis did not show a significant difference in its means for different surface treatments.

The results given in Table 4, many of which were not included in the analyses of variance, indicate that surface finish is a significant factor for some aggregates, notably the sandstone which has a greater bond when finished with a coarse grit than with a very fine grit. Among most of the carbonate rocks the surface finish generally had little effect; however, some aggregates, as shown in ANOV 1, were affected. How they would be affected however cannot be predicted. Another aggregate whose bond strength appears to be affected by surface finish is No. 15, Louisville dolomite, which had an increase in strength with a No. 1200 grit finish over the No. 60 grit surface finish. It is not possible to arrive at a statement of the effect of surface treatment which would not have exceptions even for the limited number of aggregates in this investigation.

It can be stated that, for many kinds of aggregate, the surface finish was not a significant factor under the conditions of this investigation, as the test specimens were not allowed to dry prior to testing. This prevented drying shrinkage from occurring. It is believed that surface finish may influence the amount of bond failure when drying shrinkage occurs due to increased mechanical restraint. Evidence of this was the visual observation of the bond interface of

broken, air dried mortar-aggregate cores. A visible ring could be seen around the perimeter of the interface in which the bond had been destroyed by shrinkage. It is proposed that a rough surface will offer more restraint to such shrinkage than a smooth surface.

It is for this reason that quartz, for instance, has the reputation of low bond strength in concrete. Nearly all concrete has been exposed to drying conditions and the quartz crystal facies in aggregate are smooth, thus offering little mechanical restraint to the shrinkage of concrete.

Effect of Age

The results of the bond tests shown in Table 4 were obtained at an age of 14 days. A smaller number of tests were run at an age of 28 days. The results are given in Table 5 along with corresponding results of 14 day tests. The differences between the 14 day and 28 day tests were investigated by means of the "t" test using the hypothesis that two populations with unequal variances have equal means. Table 6 is a summary of the "t" test results. It shows that only two of the differences are significant for an α of 0.05. For an α of 0.0125 only one of the differences between the 14 day and 28 day tests was significant. The one aggregate which did increase in bond strength with age was No. 6 a macro porous material containing numerous voids. From these results it was concluded that, between 14 days and 28 days, age does not have an effect on bond strength. A small number of recent results given in the Appendix indicate that the bond strength between mortar and aggregate does increase slowly

TABLE 5

RESULTS OF BOND TESTS

COMPARISON OF 14 DAY "STRENGTHS" TO 28 DAYS

All Aggregate Slabs Finished with No. 1200 Grit

Aggregate		14 Day			28 Day		
No.	Name	$\bar{\bar{X}}$	V	n_t	$\bar{\bar{X}}$	V	n_t
*6	Liston Creek (Dolomite)	53	15	26	75	13	33
9	St. Genevieve (Limestone)	58	26	33	51	21	30
11	Lincoln "Quartzite" (Sandstone)	56	18	14	40	24	21
12	Diorite	56	12	22	35	24	7
13	Harrodsburg (Limestone)	54	25	35	38	32	36
	Neat Cement	48	27	33	32	49	24

All Aggregate Slabs Finished with No. 60 Grit

Aggregate		14 Day			28 Day		
No.	Name	$\bar{\bar{X}}$	V	n_t	$\bar{\bar{X}}$	V	n_t
9	St. Genevieve (Limestone)	63	12	78	64	12	34
13	Harrodsburg (Limestone)	65	13	83	75	12	59
	Neat Cement	32	21	31	27	36	25

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* Liston Creek dolomite is very porous and the No. 1200 grit finished surface has many visible pores.

$\bar{\bar{X}}$, Mean value, pounds

V, Coefficient of Variation

n_t, Number of individual tests comprising each mean.

TABLE 6

SUMMARY OF "t" TESTS ON DIFFERENCES BETWEEN 14 DAY AND 28 DAY BOND "STRENGTHS"

Agg. No.	Name	Degree of Freedom	"t"	"t" .05	"t" .0125
No. 1200 Grit					
6	Liston Creek (Dolomite)	60	1.74	1.67	2.30
9	St. Genevieve (Limestone)	57	0.38	1.67	2.30
11	Lincoln "Quartzite" (Sandstone)	35	1.15	1.69	2.34
12	Diorite	20	1.92	1.72	2.42
13	Harrodsburg (Limestone)	72	0.87	1.67	2.29
	Neat Cement	52	0.78	1.67	2.32
No. 60 Grit					
9	St. Genevieve (Limestone)	96	0.14	1.66	2.28
13	Harrodsburg (Limestone)	139	0.85	1.66	2.27
	Neat Cement	48	0.41	1.68	2.31

with time. This is based on a small number of specimens representing two limestones at an age of approximately six months. Bond test specimens of mortar bonded to neat cement paste, likewise aged six months, did not increase appreciably in bond strength. The increase of bond strength between only the mortar and aggregate with time was indicative of chemical bond being developed between the mortar and the aggregate over a considerable time period.

Effect of Aggregate Grain Size

No aggregates were available in which the only variable between aggregates was the grain size. In order to investigate the effect of grain size within the carbonate aggregates used in this investigation, three limestones of similar lithic and chemical composition, but with a considerable range in grain size, were used in the mortar-aggregate bond test.

A group of data randomly selected from the results obtained for each aggregate and surface finish was used in ANOV 2, found in the Appendix. The means from ANOV 2 are given in Table 7 along with some of the grain size and chemical analyses of the aggregates. The results of the analysis of variance on these three aggregates and the subsequent Neuman-Keuls analysis have previously been mentioned. A significant difference in mean mortar-aggregate bond strengths exists between the kinds of aggregate, but it does not seem to be a function of grain size as the lowest bond strength was developed with the intermediate grain size. From the other aggregate properties it appears no single property has a dominant effect on the bond strength. For carbonate aggregates, grain size did not influence the mortar-aggregate bond.

TABLE 7

EFFECT OF GRAIN SIZE ON AGGREGATES OF SIMILAR COMPOSITION

Aggregate No.	Aggregate Name	Ave. Grain Size millimeters	Chemical Analysis, Percent Calcite	Dolomite	Insol. Residue	Mean Bond Strength, Pounds #60 Grit	#1200 Grit
9	St. Genevieve	fine, 0.01	88	10	2	63	67
10	St. Genevieve	medium, 0.08	92	2	5	51	51
13	Harrodsburg	coarse, 0.40	94	5	1	59	64

RESULTS OF TESTS ON CONCRETE MADE WITH SELECTED AGGREGATES

Concrete was prepared from rounded aggregate of one size of the types used in the bond tests. This concrete was cast into 2 x 2 x 11 $\frac{1}{4}$ inch beams and tested both in flexure and in indirect tension. The same type of concrete was also used in three 3 x 6 inch cylinders for compression tests. In addition, some concrete specimens of both types were made with the rock cores left from the bond strength tests, as well as with plastic rods and glass marbles.

Flexural Strength

The results of the flexural tests are summarized in Table 8. The complete results along with an analysis of variance, ANOV 4, of differences among means with one experimental factor are given in the Appendix.

The greatest flexural strengths (modulus of rupture) measured were for mortar beams without coarse aggregate, 868 psi. The results for the aggregate prepared from carbonate rocks, which had previously been tested for bond strength, were distributed from 726 to 841 psi. The differences in these means from 868 to 726 psi are not significant as shown by the analysis of variance ANOV 4 in the Appendix.

The beams prepared with glass marbles gave an interesting comparison of the effect of surface treatment. Marbles roughened with No. 60 grit resulted in flexural strength of 578 psi compared

TABLE 8

RESULTS OF FLEXURAL TESTS

2 x 2 x 11¼ inch beams, w/c = 0.60, s/c = 2.50

Aggregate		Flexural Test		Particle Shape*	
No.	Name	$\bar{S}_f$, psi	n_t	L/W	W/T
2	Jeffersonville (Dolomite)	387	2	1.33	1.81
6	Liston Creek (Dolomite)	803	6	1.41	1.52
9	St. Genevieve (Limestone)	780	4	1.39	1.74
10	St. Genevieve (Limestone)	763	2	1.34	1.53
13	Harrodsburg (Limestone)	841	4	1.36	1.69
14	Geneva (Dolomite)	726	3	1.37	1.55
15	Louisville (Dolomite-Limestone)	760	4	1.38	1.59
16	Gravel (Quartzite)	639	4	1.30	1.42
	Marbles, untreated (glass)	405	4	1.00	1.00
	Marbles, No. 60 grit finish (glass)	578	4	1.00	1.00
	Plexiglas rods	304	2	1.12	1.00
	Mortar only	868	5		

$\bar{S}_f$, Mean Modulus of rupture, psi as determined by ASTM Designation: C-293

n_t, Number of tests on which the mean is based

L, The mean greatest dimension of pieces of aggregate.
W, The mean intermediate dimension of pieces of aggregate.
T, The mean smallest dimension of pieces of aggregate.
w/c, Water-cement ratio
s/c, Sand-cement ratio

*Based on a sample of 50 pieces of aggregate.

to 405 psi for smooth untreated marbles. These means are significantly different from those of the carbonate aggregates, as shown by the analysis of variance ANOV 4. in the Appendix.

Approximately nine beams were prepared from the rock cores remaining from the bond tests. These specimens gave flexural values of 440 to 717 psi (see Appendix) which were lower than the values obtained from the corresponding rounded and washed aggregate. This difference is attributed to differences in particle shape and the smoother surface of the cores.

Two beams were made with plexiglas rods for the coarse aggregate. The low mortar-plexiglas bond strength enabled a comparison to be made of flexural strength with aggregates of similar shape but with greatly different mortar bond strengths. The average flexural strength of the plexiglas aggregate concrete was 304 psi, roughly half as great as the average strength of concrete with rock cores for its aggregate.

The method of failure of the plexiglas aggregate concrete was different than for the other concretes although the method of test was identical. It was not possible to maintain a uniform rate of loading. Two or three times during the course of the test, the load dropped off as if failure occurred but almost immediately, the load increased and the specimen continued to sustain increased load until this action was repeated or, eventually, true failure occurred. Apparently the concrete was failing locally allowing sufficient deflection to bring about a decrease in total load. After the maximum load was obtained, the concrete did not break apart but

continued to deform with many local cracks developing. The deformation was accompanied by a slow loss in load.

This unusual behavior of the plexiglas concrete is attributed to several causes. The early drops in load are believed to have been caused by the failure of a very small bond developed over large areas of some of the plexiglas rods. The rods continued to provide an interlocking action, and due to the low modulus of elasticity of the plexiglas, the rods could bend, something that the mineral aggregates do not do as compared to a material such as plexiglas.

The low flexural strength resulted from two sources, the first is the low bond strength and the second is the lower modulus of elasticity of the plexiglas. A majority of the concrete was composed of plastic, which has a much lower modulus of elasticity than the mineral aggregates. It has been shown (15) that a low modulus of elasticity in the aggregate results in a relatively low modulus for the resulting concrete. The flexural strength of the concrete would likewise be reduced by aggregate of lower modulus of elasticity.

Indirect Tensile Strength

The results of the indirect tensile test are not reported because it was believed that the effect of bond strength was not accurately given due to the size of these particular test specimens. The small specimen size relative to the size of the aggregate and the tight nesting of the coarse aggregate against the side of the specimen, as well as each other, induced sufficient complexities and restraints in the distribution of the stresses under the type of loading used to make the results less indicative of the tensile strength than if

a larger beam had been used (17). The results did not appear to conflict with those of the flexure tests.

Compressive Strength

The results of the compression tests on the 3 x 6 inch cylinders are given in Table 9. The individual test results are given as is the mean of the results for each different aggregate. The inflection stress, S_I, is the stress when the number of cracking impulses commences to increase at a much greater rate, previously demonstrated in Figure 9. The inflection ratio is the ratio of the inflection stress to the ultimate compressive strength. The compressive strengths range from a low of 1,260 psi for concrete made with plexiglas cores to 4,990 psi with a mortar cylinder containing no coarse aggregate. All of the rounded natural aggregate had mean compressive strengths between 3,140 and 3,420 psi. The natural aggregates used in the compression tests were all carbonate rocks.

Only one cylinder was made with cores left from the bond tests. Its compressive strength was considerably lower than the results for the rounded natural aggregate. This was attributed to the particle shape of the cores.

The two cylinders made with plexiglas cores both had low strengths. The failed cylinders were carefully examined, and it was found that bond failure occurred or could easily be made to occur next to all plexiglas cores with one exception. Whenever a plexiglas core was so oriented that the axis of the core was nearly parallel to the axis of the cylinder, the top (as it was cast) of the cores was bonded to the mortar. The ends of the cores had been cut with a

TABLE 9

RESULTS OF COMPRESSION TESTS

INCLUDING CRACK IMPULSE MEASUREMENTS

3 x 6 inch cylinders

Aggregate						Inflection			
No.	Description	L/W	W/T	Cyl. No.	S_c,psi	Stress S_I,psi	Ratio %	S_c psi	S_I psi
2	Jeffersonville	1.330	1.811	128	3,126	1,272	40.7		
	(Dolomite)	1.330	1.811	129	3,281	1,697	51.7	3250	1410
		1.330	1.811	130	3,338	1,272	38.1		
6	Liston Creek	1.409	1.518	115	3,055	2,744	78.7		
	(Dolomite)	1.409	1.518	116	3,253	1,666	51.2	3230	1940
		1.409	1.518	118	3,380	1,413	41.8		
9	St. Genevieve	1.386	1.735	114	3,649	1,900	73.6		
	(Limestone)	1.386	1.735	117	2,730	1,305	47.8	3140	1520
		1.386	1.735	119	3,027	1,344	44.4		
10	St. Genevieve	1.337	1.533	134	3,479	1,131	32.5		
	(Limestone)	1,337	1.533	135	3,352	1,415	42.2	3420	1460
		1,337	1,533	136	3,423	1,838	53.7		
13	Harrodsburg	1.558	1.690	126	3,536	2,263	64.0	3250	2260
	(Limestone)	1.558	1.690	127	2,956	2,261	76.5		
14	Geneva	1.372	1.553	131	2,999	1,697	56.6		
	(Dolomite)	1.372	1.553	132	3,395	2,403	70.8	3150	1930
		1.372	1.553	133	3,677	1,699	46.2		
15	Louisville	1.385	1.590	120	3,182	1,699	53.4		
	(Dolomite-	1.385	1.590	121	3,005	2,353	78.3	3150	1870
	Limestone)	1.385	1.590	122	3,253	1.555	47.8		
	Marbles	1.00	1.00	104	1,641			1770	1340
	(Smooth)	1.00	1.00	105	1,711	1,414	82.6		
		1.00	1.00	106	1,966	1,273	64.7		
	Marbles	1.00	1.00	107	2,546	1,980	78.0	2550	1760
	(Rough # 60)	1.00	1.00	108	2,829				
		1.00	1.00	109	2,829	1,545	56.0		
		1.00	1.00	137	1,981				
	Mortar			110	4,653	1,697	36.5	4310	2810
				111	4,993	2,150	43.1		
				112	*				
				113	*				
				123	3,791	1,781	47.0		
				124	3,890	1,186	30.5		
				125	4,215	**			

TABLE 9, continued

RESULTS OF COMPRESSION TESTS

INCLUDING CRACK IMPULSE MEASUREMENTS

3 x 6 inch cylinders

Aggregate No.	Description	L/W	W/T	Cyl. No.	S_c, psi	Inflection Stress S_I, psi	Inflection Ratio %	S_c psi	S_I psi
	Plexiglas			138	1,584	N/A		1420	
				139	1,259	N/A			
	Cores of #13LS			140	2,299	N/A		2300	

* Discarded due to defective molds.

** Impulse counting apparatus failed.

band saw, and the resulting surface was apparently such that it developed a bond when mortar was placed over the surface. Conditions beneath the rod were apparently sufficiently different so as to not develop an appreciable bond.

Comparison of Compression Test Results on Cylinders to Flexural Test Results on Beams

An overall comparison of the two types of tests indicated that the flexural and the compression tests were in agreement as anticipated from the results of other investigations (18). High flexural strength for example was indicative of high compressive strength.

All of the tests demonstrated differences in the results which are attributable to the coarse aggregate. The influence of the mortar bond strength and other aggregate properties are discussed in the following two sections.

DISCUSSION OF THE MORTAR-AGGREGATE BOND TEST RESULTS COMPARED TO CONCRETE STRENGTH TEST RESULTS

Bond strength was determined for two surface conditions, the results having been previously discussed. Because of the general lack of a significant difference being developed between the two different surface finishes on different aggregates, the average of the bond strengths for aggregate slab surfaces finished with No. 60 grit and with No. 1200 grit has been used for comparison with the concrete test results.

Effect of Bond on Flexural Strength

The flexural test results, given in Table 8, have been plotted versus the 14 day mortar-aggregate bond strength of the coarse aggregate, in Figure 11. The resulting distribution of points indicated that the bond strength was indicative of the flexural strength. Some inconsistencies occurred, but they did not disturb the overall trend.

The values of bond strength for both plexiglas and mortar were not determined by bond tests for such tests could not be performed using these materials. The plexiglas formed too weak a bond to permit measurement, however, some bond did exist. A value of 10 pounds was selected as an estimate. The bond strength for mortar without aggregate was similarly selected for it was known that the mortar had a greater bond strength than most of the bonds, however, some

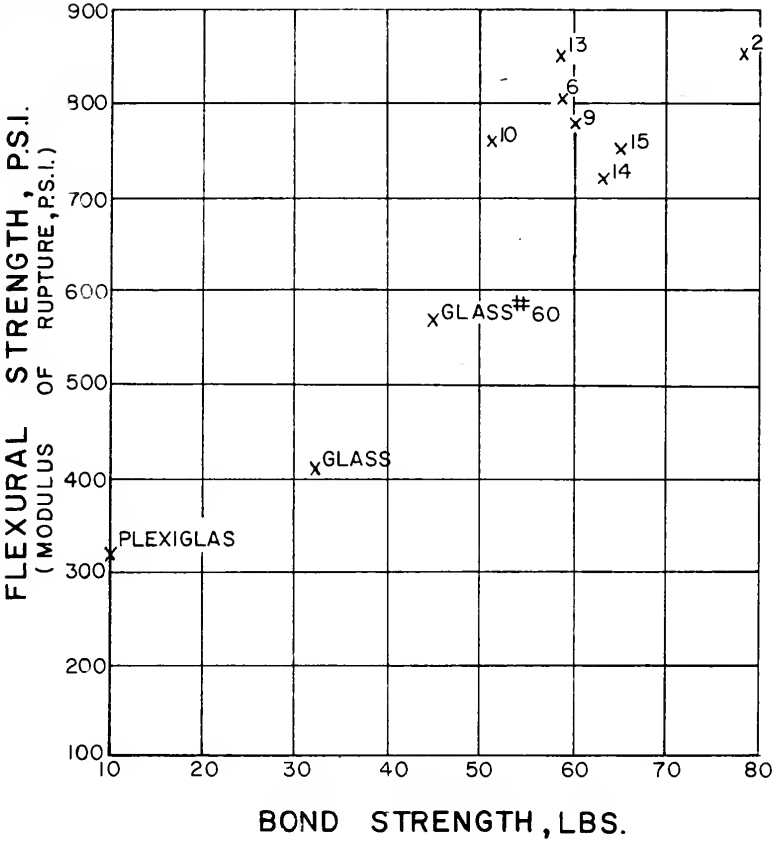

FIGURE II. BOND STRENGTH VERSUS FLEXURAL STRENGTH

mortar failures occurred with aggregates having the higher bond strengths so it was assumed that the mortar's bond strength was not a great deal higher, probably 80 pounds or slightly higher.

Insofar as feasible, bond strength was the only variable, however, many other factors pertaining to the coarse aggregate, and discussed elsewhere, may have influenced the strength of the concretes. If, for a given type of material, differences in bond strength of that material to a mortar of a given composition is primarily dependent upon surface finish, one would expect such differences to be reflected in the strength of concretes made with those materials if bond affects strength. The glass marble aggregates were used for two different concretes with the only variable being the surface finish. It was gratifying to see that the relative positions for these concretes, on Figure 11, agreed well with the other results. Other factors being the same, the coarse aggregate with the greatest mortar-aggregate bond strength will result in the greater flexure and compressive strength for 14 day old concrete.

If the bond strength values obtained in this investigation are converted to the modulus of rupture, the results of tests on concrete may be compared with those obtained by Alexander et al, (4). The results agree reasonably well even though the type of bond test specimen was not the same.

Effect of Bond on Compressive Strength

The results of the compression tests on 3 x 6 inch cylinders, given in Table 6, have been plotted versus the average mortar-aggregate bond strength of the coarse aggregate, in Figure 12. The

values of bond strength for both the plexiglas and the mortar were again estimated.

The plot of the compressive test results versus the mortar-aggregate bond strength of the coarse aggregate shows that the influence of bond strength on compressive strength is similar to its influence on flexural strength. The carbonate aggregates produced concrete whose mean compressive strengths vary between 3,150 and 3,420 psi. These concretes are closely grouped compared to the mean compressive strength obtained for concretes made with marbles, 2550 psi for roughened and 1773 psi for the smooth marbles. The influence of bond is apparent, over the range of values included, as it also is when the compressive strength of the mortar cylinders without coarse aggregate, 4310 psi, is included with the estimated bond strength for the mortar. The concrete strength was greater for concrete produced with a coarse aggregate having a greater mortar-aggregate bond strength if other factors remain unchanged.

The compressive strengths of concrete obtained with coarse aggregates having various bond strengths were compared to the results of Alexander et al, (4). The bond strengths were expressed as modulus of rupture and the paste strength was estimated by interpolation from available laboratory data on the cement used in this investigation. The concrete compressive strength in this investigation was considerably higher than that obtained using Alexander's expression, $B + 2P$, when B equals the bond strength and P equals the paste strength. It is possible that the method of preparing the coarse aggregate and the compression specimens was partially responsible for the discrepancy.

The plot of the inflection stresses obtained from the cracking impulse count data did not appear to be greatly influenced by the bond strength between the coarse aggregate and the mortar. The inflection ratio was, therefore, generally greater for concretes whose coarse aggregate had the lower mortar-aggregate bond strength.

The data were not sufficient to be conclusive, but did suggest the following hypothesis for concrete failure.

> When the mortar-aggregate bond strength characteristic is the only variable among aggregates, i.e., the aggregates have identical size, shapes, physical properties, etc., concrete made with these coarse aggregates and a mortar will form microcracks at a compressive stress that is primarily dependent upon the properties of the mortar. The amount of microcracking, as reflected by the increase in number of microcracks to reach incipient failure is dependent upon the mortar-aggregate bond of the coarse aggregate.

The microcracks (separations) occurring in the concrete, as reflected by the impulse counts, take place at approximately the same rate, as indicated in Figure 9, once they commence. These microcracks occur in order to relieve stresses developed within the concrete. The initiation of cracking is dependent upon the formations of sufficient stress concentrations to cause microcracking or separations. If an increment of stress will be relieved by one crack, regardless of the mortar-aggregate bond involved, it becomes apparent that the area of separation required to relieve the increment of

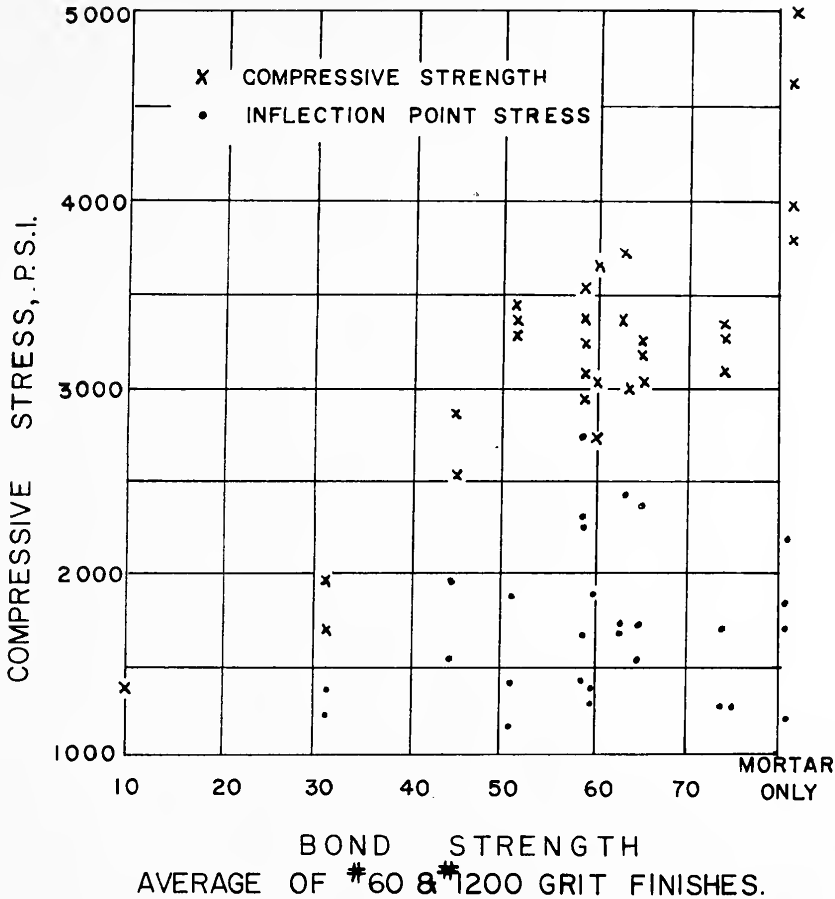

FIGURE 12. COMPRESSIVE STRESSES VERSUS BOND STRENGTH AT FAILURE AND INFLECTION POINTS.

stress will be greater for concrete having the lower mortar-aggregate bond. As the load on the concrete continues to be increased, the number of stress increments requiring relief will remain the same regardless of the mortar-aggregate bond. The total cumulative area of separation, however, will be greater for the concrete with the lower bond strength thus the condition in which the overall strength of the specimen is insufficient to withstand the load is reached at lower applied load for the concrete with the lower bond strength.

This hypothesis cannot be applied to the work of Jones and Kaplan (4) inasmuch as their aggregates were intentionally different in variables other than the bonding characteristics. One of their observations is nevertheless worthy of note and appears to be in agreement with the above hypothesis. "The relation between the flexural strength of concrete and the stress at which cracks first appear in compression was independent of the type of coarse aggregate" (4). This suggests that other variables such as shape and physical properties may not greatly influence the initiation of microcracking.

INFLUENCE OF AGGREGATE PROPERTIES

The results of the investigation clearly showed that aggregate properties did influence both bond and concrete strength. The differences in concrete strength cannot all be attributed to variations of bond strength and, no doubt, were influenced by a number of other factors.

Aggregate Properties and Mortar-Aggregate Bond Strength

Comparing the aggregate properties, given in Table 1, to the bond strength of the aggregate, the striking feature was the apparent lack of any factor which seemed to be correlated with the bond strength. The grain size was not significant, chemical composition among the carbonates did not appear to influence the results nor did the various physical properties such as the modulus of elasticity, absorption or specific gravity. The data, however, were insufficient to enable a firm overall statement to be made regarding the influence, or the lack of influence, of these factors.

Aggregate Properties and the Strength of Concrete

None of the aggregate properties mentioned with regard to their lack of apparent correlation on bond appeared to have any influence on the concrete strength. This is not to imply that properties such

as the modulus of elasticity were not associated with strength, it means only that such factors were offset by others that were not identified or controlled.

The particle shape of the aggregate used in the concrete was evaluated by finding the average length, L, width, W, and thickness, T, of a random sampling of fifty particles from each aggregate. The results are given in Table 1 in the form of L/W and W/T. The particle shape appears to influence the concrete strength as shown in Figure 13, where the particle shape is represented by the sum of the average L/W and W/T for each aggregate.

It should be noted that the effect of bond strength and of particle shape with the low bond strength aggregates, i.e., the glass marbles and the plexiglas, reinforced each other; thus Figure 13 cannot be considered to illustrate the effect of particle shape with all other properties being constant. It must also be noted that the concrete in this investigation was somewhat different from normal concrete due to the one size coarse aggregate and the method of casting the test specimens by forcing as much aggregate as possible into the specimen. The interlocking action may be greater in such a concrete than in one made by normal procedures thus exaggerating the effect of particle shape. In a well compacted concrete, tested in flexure, aggregate with the higher L/W and W/T value is expected to give a higher strength than obtained with more spherical aggregate.

The results of beams made with the remains of rock cores are not shown on Figure 13 since only one beam could be prepared for each aggregate. The results for these isolated tests are given in

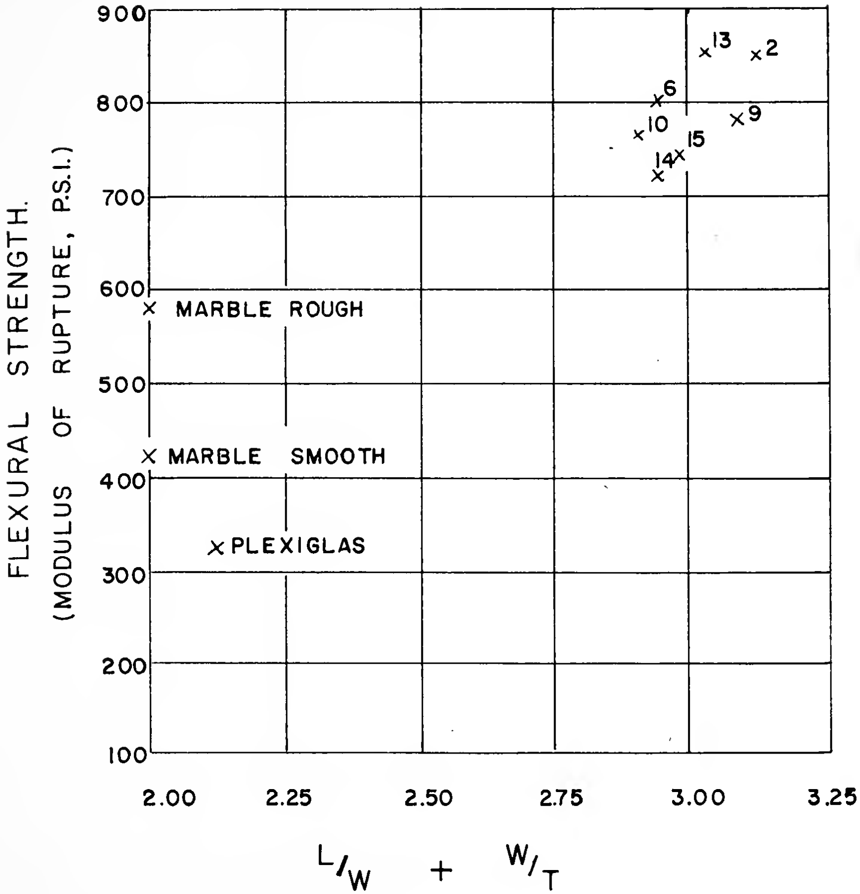

FIGURE 13. EFFECT OF PARTICLE SHAPE ON FLEXURAL STRENGTH.

Table 10 in the Appendix. If plotted on Figure 13, they agree quite well with the apparent trend.

Particle shape appeared to have less influence on the compressive strength of the 3 x 6 inch concrete cylinders than on the flexural and indirect tensile strengths of the beam tests. Figure 14 shows the compressive strengths versus the evaluation of particle shape.

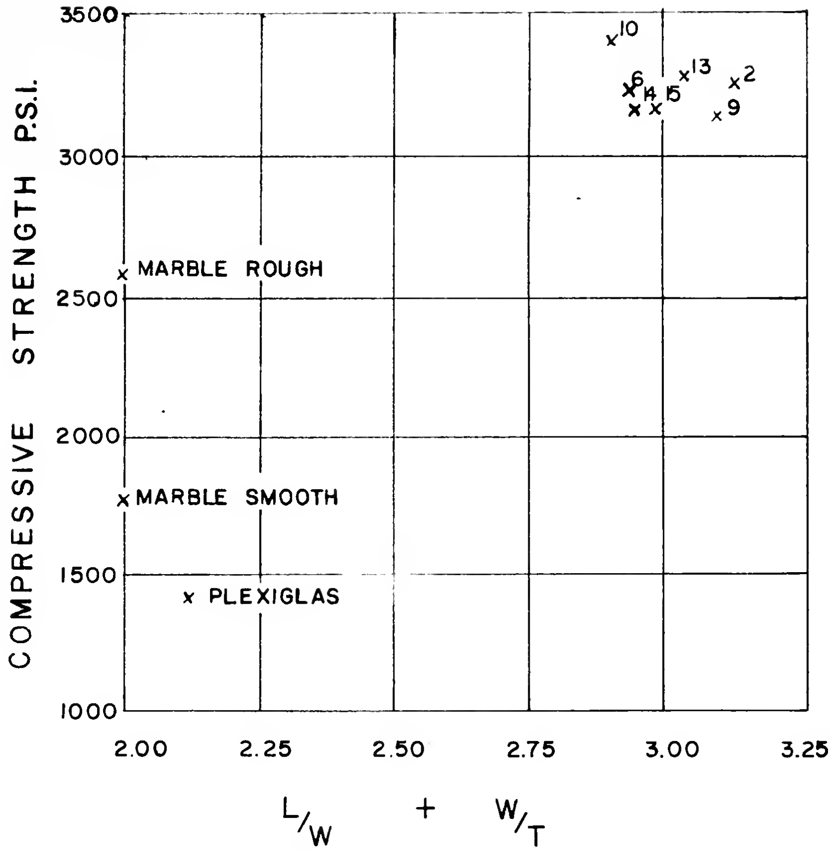

FIGURE 14. EFFECT OF PARTICLE SHAPE ON COMPRESSIVE STRENGTH.

SUMMARY AND CONCLUSIONS

The bond strength between portland cement mortar and coarse aggregate was evaluated by developing a test method which was used on a variety of aggregates. Various factors which might influence the bond were investigated in order to obtain information on the nature of the mortar-aggregate bond. A summary of the findings follows:

1. A test was developed to measure bond between an aggregate surface and portland cement mortar. The test has a high variability as do other bond tests that have been used to measure mortar-aggregate bond. However, the method developed allows a large number of specimens to be prepared without undue effort and with a great deal of physical uniformity between the test specimens.
2. The type of aggregate influenced the amount of mortar-aggregate bond. The highest bond strengths were obtained with quartzite.
3. Grain size of the aggregate was not related to the bond strength.
4. No relationship was found between the measured chemical and physical properties of the aggregates and bond strength.
5. For the condition of no drying (i.e., constant saturation) of mortar-aggregate bond specimens in this investigation,

the surface finish usually did not affect the bond strength. It is entirely possible that under conditions of partial or complete drying, surface finish may influence the amount of bond destroyed by shrinkage. It is proposed that a rough surface will offer considerable restraint to drying shrinkage.

6. The bond strength did not change appreciably between the ages of 14 and 28 days. A small number of tests made on specimens at approximately six months of age indicate a significant increase in bond strength between mortar and aggregate at this later age.

7. Concrete was made with several of the aggregates on which mortar-aggregate bond strength determinations had been made. The concrete aggregate was rounded to minimize the effect of particle shape and texture. One-size aggregate was used to eliminate the effect of gradation. Flexural and compressive strength tests were performed on the concrete. The results were compared to those of the bond test. A summary of the findings follows:

    a. The flexural strength of the concrete was significantly different for aggregates having greatly different mortar-aggregate bond strengths (i.e., limestone, glass and plastic).

    b. The concretes made with natural aggregates, having bond strengths representative of all natural aggregates tested, did not have a significant

difference in flexural strength.

c. The compressive strength of the concrete was influenced in a manner similar to that observed in the case of flexural strength. Appreciable differences in compressive strength were obtained only for those aggregates which had greatly different bond strengths.

d. The impulses, presumably caused by microcracking in the concrete under compression, were counted electronically. The initiation of these impulses and the stress present at the initiation was obtained during the compression tests of the concrete. A hypothesis for the failure of concrete under compression was proposed utilizing the mortar-aggregate bond strength to explain the concrete's behavior.

The following conclusions are made on the basis of results obtained in this investigation.

1. The amount of mortar-aggregate bond strength is not easily relatable to mineralogical, chemical or physical properties of the aggregate.
2. Differences in mortar-aggregate bond strength do exist, but for a wide variety of natural aggregates the differences in bond strength measured under the test conditions of this investigation are not very large.

3. Bond strength can greatly influence the flexural and compressive strengths of concrete if the differences in bond strength are very large. For concrete which has not been allowed to dry, which was made with a wide variety of natural aggregates, and in which the effect of shape, texture and gradation is minimized, the influence of mortar-aggregate bond is not significant.

RECOMMENDATIONS

1. The effect of age on the portland cement mortar-aggregate bond over a considerable period of time should be determined and compared with the increase in mortar strength over a similar period of time.

2. The amount of restraint to drying shrinkage of the mortar caused by surface roughness and its effect on the bond strength of dry mortar-aggregate interfaces would provide information pertinent to concrete under many natural conditions.

3. The effect of the restraint to drying shrinkage caused by surface roughness and its effect on bond strength should be evaluated where the mortar-aggregate interfaces are subjected to pressures of the range experienced in concrete.

4. Study of the influence of particle shape, with mortar-aggregate bond strength and gradation being constant, would more clearly demonstrate particle shapes' effect on concrete.

5. The technique for measuring microcracking within concrete offers the possibility of being a very useful tool in investigating the behavior of concrete. It warrants further work to improve the apparatus and the methods employed to pick up and count the impulses.

6. The proposed hypothesis of the compression failure of concrete, considering bond strength and microcracking warrants further investigation.

7. Utilizing the microcrack impulse counting technique, further studies of the effect of bond strength and indeed of the very nature of both fatigue and creep of concrete may be made.

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APPENDIX

APPENDIX

TABLE 10

RESULTS OF FLEXURAL TESTS ON CONCRETE BEAMS

2 x 2 x 11¼ inches

Aggregate Number and Description	Beam No.	Coarse Agg. in Beam, % by Volume	**** L/W	**** W/T	S_f, psi
Rounded and Washed, Passing 1 in. Retained 3/4 in.					
2 Jeffersonville (Dolomite)	11	56.2	1.330	1.811	819
	12	57.3	1.330	1.811	855
	27	58.2	1.330	1.811	782
	28	58.2	1.330	1.811	656
6 Liston Creek (Dolomite)	32		1.409	1.518	888
	33		1.409	1.518	833
	44		1.409	1.518	863
	45		1.409	1.518	883
	53		1.409	1.518	661
	54		1.409	1.518	690
9 St. Genevieve (Limestone)	17	58.9	1.386	1.735	770
	18	60.4	1.386	1.735	781
	30	58.7	1.386	1.735	***
	31	60.2	1.386	1.735	792
	41	59.38	1.386	1.735	779
10 St. Genevieve (Limestone)	9	57.6	1.337	1.533	765
	10	56.2	1.337	1.533	761
	23	57.8	1.337	1.533	963
	24	57.8	1.337	1.533	787
13 Harrodsburg (Limestone)	15	59.6	1.558	1.690	934
	16	59.8	1.558	1.690	820
	34	60.0	1.358	1.690	847
	35	60.0	1.358	1.690	766
14 Geneva (Dolomite)	13	59.6	1.372	1.553	843
	14	59.6	1.372	1.553	780
	29	59.9	1.372	1.553	543
15 Louisville (Dolomitic Limestone)	21	56.4	1.385	1.590	819
	22	56.4	1.385	1.590	728
	39	57.1	1.385	1.590	653
	40	57.3	1.385	1.590	842
Marbles (Smooth)	25	53.2	1.00	1.00	360
	26	53.1	1.00	1.00	389
	42	54.49	1.00	1.00	446
	43	56.33	1.00	1.00	427

TABLE 10, continued

RESULTS OF FLEXURAL TESTS ON CONCRETE BEAMS

2 x 2 x 11¼ inches

Aggregate Number	Description	Beam No.	Coarse Agg. in Beam, % by Volume	**** L/W	**** W/T	S_f, psi
	Marbles (rough #60 Grit)	50	54.07	1.00	1.00	654
		51	53.80	1.00	1.00	601
		55	51.42	1.00	1.00	538
		56	52.89	1.00	1.00	518
	Mortar Only (No Coarse Aggregate)	38	N/A	N/A	N/A	964
		46	N/A	N/A	N/A	803
		47	N/A	N/A	N/A	961
		48	N/A	N/A	N/A	827
		49	N/A	N/A	N/A	786
	Cores or Cylinders					
	Plexiglas	52				231
		57				378
2	see 2 above	59				
6	see above	60				440
7&8	Quartzite	58				159*
9	see above	61				535
11	Sandstone, carbonate bonding	62				512
13	see above	63				717
14	see above	65				692**
15	see above	64				489**

**** Based on a sample of 50 pieces of aggregate.

*** Broke beam when removed from mold

** Results Obtained on Beams less than 11¼ inch length

* Sudden Loading to Failure Disregard Value

S_f = Modulus of rupture, psi, as determined by ASTM Designation: C293.

L = The mean greatest dimension of pieces of aggregate.

W = The mean intermediate dimension of pieces of aggregate.

T = The mean smallest dimension of pieces of aggregate.

ANALYSIS OF MORTAR-AGGREGATE BOND TEST RESULTS

ANOV 1

(Analysis of Variance)

Plan

	j_1	j_2	j_3
i_1	i_1j_1	i_1j_2	i_1j_3
i_2	i_2j_1	i_2j_2	i_2j_3

i - Surface Finish

j - Aggregate

i_1 - Finished No. 60 Grit

i_2 - Finished No. 1200 Grit

j_1 - C4LS-1 (Aggregate No. 4) (Not used in other portions of this investigation)

j_2 - C2LS-1 (Aggregate No. 2)

j_3 - C6LS-1 (Aggregate No. 6)

Abbreviated ANOV Table

Source	Deg. of Freedom	SS	MS	EMS	F	$F_{.05}$
i Surface Finish	1	2.20587	2.2	$\sigma_\epsilon^2+51\,\phi_i$	0.019	3.95
j Aggregate Type	2	2032.3723	1016.2	$\sigma_\epsilon^2+34\,\phi_j$	8.64	3.10
ij	2	4287.588	2143.8	$\sigma_\epsilon^2+17\,\phi_{ij}$	18.23	3.10
error	96	11186.8196	106.1	σ_ϵ^2		

- Variance component due to experimental error

i - Variance component due to i (surface treatment)

j - Variance component due to j (aggregate type)

ij - Variance component due to ij (interaction)

Neuman-Kuels Analysis for ANOV 1

The Means Arranged in Decending Order

A	B	C	D	E	F
68.4	67.2	64.0	63.5	54.9	46.0
i_2j_2	i_2j_3	i_1j_1	i_1j_2	i_1j_3	i_2j_1

k = 6 n = 17 d.f. = 96

$s^2 = 117.52$ $s_{\bar{y}} = \sqrt{\frac{117.57}{17}} = 3.20$

	Upper Percentile Points (19)		$s_{\bar{y}}$	R value
R_6 =	4.14	x	3.20 =	13.3
R_5 =	3.94	x	3.20 =	12.6
R_4 =	3.71	x	3.20 =	11.9
R_3 =	3.35	x	3.20 =	10.7
R_2 =	2.81	x	3.20 =	9.0

Differences between means with (R value)

	F	E	D	C	B
A	22.4 (13.3)	13.5 (12.6)	4.9 (11.9)	4.0 (10.7)	1.2 (9.0)
B	21.2 (12.6)	12.3 (11.9)	3.7 (10.7)	3.2 (9.0)	
C	18.0 (11.9)	9.1 (10.7)	0.5 (9.0)		
D	17.5 (10.7)	8.6 (9.0)			
E	8.9 (9.0)				

Summarization: A B C D E F

Means A, B, C and D are different from mean F and means A and B are also different from E as well as F.

ANALYSIS OF MORTAR-AGGREGATE BOND TEST RESULTS

ANOV 2

(Analysis of Variance)

Plan

	j_4	j_5	j_6
i_1	i_1j_4	i_1j_5	i_1j_6
i_2	i_2j_4	i_2j_5	i_2j_6

i - Treatments

j - Rocks

i_1 - Finished No. 60 Grit

i_2 - Polished thru No. 1200 Grit

j_4 - C9LS-1 (Aggregate No. 9)

j_5 - C10LS-1 (Aggregate No. 10)

j_6 - C13LS-1 (Aggregate No. 13)

Abbreviated ANOV Table

Source	Deg. of Freedom	SS	MS	EMS	F	$F_{.05}$
i	1	244.745	244.745	$\sigma_\epsilon^2 + 51\,\phi_i$	1.88	3.95
j	2	3496.293	1748.147	$\sigma_\epsilon^2 + 34\,\phi_j$	13.45	3.10
ij	2	85.314	42.657	$\sigma_\epsilon^2 + 17\phi_{ij}$	0.32	3.10
error	96	12475.056	129.94	σ_ϵ^2		

- Variance component due to experimental error
i - Variance component due to i (surface treatment)
j - Variance component due to j (aggregate type)
ij - Variance component due to ij (interaction)

Neuman-Kuels Analysis for ANOV 2

The means for the two finishes, i_1 and i_2, have been combined since significant differences were not found between surface finishes or as on interaction between surface finish and aggregate type.

The Means Arranged in Decending Order

A	B	C
130.5	123.5	102.9
j_4	j_6	j_5

$k = 3$ $n = 34$ $d.f. = 96$ $s^2 = 129.94$ $s_{\bar{y}} = \sqrt{\frac{129}{34}} = 1.95$

Upper Percentile Points (19)

				$s_{\bar{y}}$		R value
R_3	=	3.35	x	1.95	=	6.53
R_2	=	2.81	x	1.95	=	5.48

Differences between Means with (R value)

	C	B
A	27.6 (6.5)	7.0 (5.5)
B	20.6 (5.5)	

Summarization:

A B C

Means A, B and C are each different from the other.

ANALYSIS OF MORTAR-AGGREGATE BOND TEST RESULTS

ANOV 3

(Analysis of Variance)

Plan

	$\underline{j_7}$	$\underline{j_8}$
i_1	i_1j_2	i_1j_8
i_2	i_2j_7	i_2j_8

j_7 - C7 Qz - 1 (Aggregate No. 7)

j_8 - C8 Qz - 1 (Aggregate No. 8)

Abbreviated ANOV Table

Source	Deg. of Freedom	SS	MS	EMS	F	$F_{.05}$
i	1	6.485	6.485	$\sigma_\epsilon^2+34\,\phi_i$	0.037	4.00
j	1	2557.191	2557.191	$\sigma_\epsilon^2+34\,\phi_j$	14.693	4.00
ij	1	430.015	430.015	$\sigma_\epsilon^2+17\,\phi_{ij}$	2.47	4.00
error	64	11138.822	174.04	σ_ϵ^2		

- \- Variance component due to experimental error
- i - Variance component due to i (surface treatment)
- j - Variance component due to j (aggregate type)
- ij - Variance component due to ij (interaction)

Neuman-Kuels Analysis for ANOV 3

The Means Arranged in Decending Order

A	B	C	D
79.2	77.8	72.0	66.4
i_1j_7	i_2j_7	i_1j_8	i_2j_8

$k = 4$ $n = 17$ $d.f. = 64$

$s^2 = 174$ $s_{\bar{y}} = \sqrt{\frac{174.04}{17}} = 3.20$

Upper Percentile Points (19)		$s_{\bar{y}}$	R value
R_4 =	3.74	x 3.20 =	12.0
R_3 =	3.46	x 3.20 =	10.9
R_2 =	2.83	x 3.20 =	9.1

Differences between means with (R value)

	D	C	B
A	12.8 (12.0)	7.2 (10.9)	1.4 (9.1)
B	11.4 (10.9)	5.8 (9.1)	
C	5.6 (9.1)		

Summarization: A B C D

Means A and B are different from the mean of D

ANALYSIS OF MORTAR-AGGREGATE BOND TEST RESULTS

ANOV 4

(Analysis of Variance)

Differences among several means composed of an unequal number of replicates with one experimental factor.

Plan:

$\underline{m_1}$	$\underline{m_2}$	$\underline{m_3}$ . . .	$\underline{m_k}$
x_1	x_1	x_1	x_1
x_2	x_2	x_2	.
.	.	x_3	.
.	.	.	.
x_{n_1}	x_{n_2}	x_{n_3}	x_{n_k}

Procedure:

$$Q = \Sigma(\Sigma X^2) - \frac{\Sigma(\Sigma X)^2}{N}$$

$$Q_c = \Sigma\frac{(\Sigma X)^2}{N_k} - \frac{\Sigma(\Sigma X)^2}{N}$$

$$Q_e = Q - Q_c$$

N = Number of replicates

$$\sigma_c^2 = \frac{Q_c}{k-1} \qquad \sigma_e^2 = \frac{Q_e}{N-k}$$

$$F_c = \frac{\sigma_c^2}{\sigma_e^2}$$

d.f. for σ_c = k - 1

d.f. for σ_e = N - k

Results: a. Among concrete made from rounded and washed aggregate and including marbles and mortar only

$$F_c = 11.16, \; F_{.05} = 2.20, \; F_{.01} = 3.05$$

A significant difference exists between some of the mean flexure strengths for the concretes.

b. Among concretes made from rounded and washed carbonate aggregate.

$$F_c \quad 1.0$$

No significant difference exists between the means of the flexural strength for the concretes.

c. Among concretes made from carbonate aggregate and mortar only

$$F_c = 1.01 \qquad F_{.05} = 2.39$$

No significant difference exists between the means of the flexural strength for the concrete.

d. Among concretes made from carbonate aggregate and rough marbles

$$F_c = 3.40 \qquad F_{.05} = 2.40 \qquad F_{.01} = 3.47$$

A significant difference does exist between the mean of the rough marbles and the carbonate aggregate concrete.

DETERMINATION OF THE MODULUS OF ELASTICITY FOR THE COARSE AGGREGATE

The moduli of elasticity for various coarse aggregates are given in Table 2. These values were determined by means of stress-strain determination conducted with a testing machine and a Tuckerman optical strain gage.

Rock specimens were cored perpendicular to each other and with one of the orientations parallel to the bedding plane of the rock. Figure 15 shows some of these cores in the position from which they were cut. These cores were capped to insure that their ends were parallel.

Figure 16 shows a core, with a Tuckerman gage mounted on it, in the testing machine. Only one optical strain gage was used because of the small diameter of the core. To correct for bending, each core was tested twice with the Tuckerman gage being on opposite sides of the cylinder during each test. The average strain was used along with the stress to enable a stress-strain curve to be drawn.

When perpendicular cores produced different stress-strain curves, an average value was used and the modulus of elasticity was determined from this curve.

The specimens were not loaded until failure but were kept considerably below failure to insure that no damage would occur to the strain gage. They were loaded over a range of loads from 2500 to

FIGURE 15. CAPPED ROCK CORES IN THE POSITION FROM WHICH THEY WERE CUT.

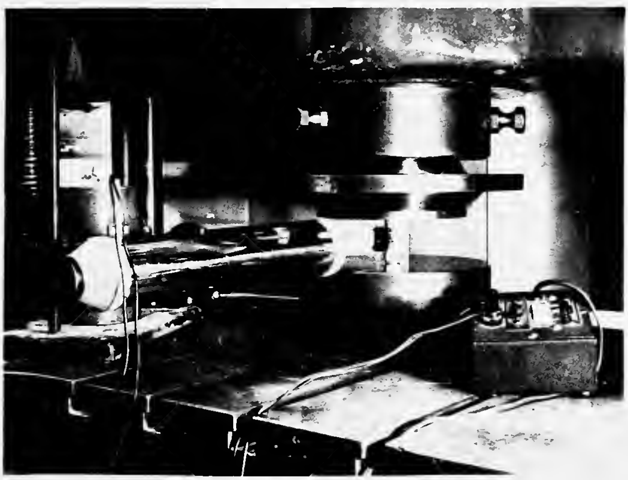

FIGURE 16. A CORE WITH A TUCKERMAN GAGE MOUNTED ON IT SET UP FOR TESTING

5500 psi depending upon the expected maximum load. Over this range of loading the stress-strain curves were essentially straight lines. The specimens were loaded in increments of 275 to 550 psi, with strains gage readings taken between loadings.

TABLE 11

SUMMARY OF RECENT MORTAR-AGGREGATE BOND TEST RESULTS AT APPROXIMATELY SIX MONTHS AGE

$w/c = 0.60, \quad s/c = 2.50$

Aggregate No.	Name	$\bar{X}$	V	M_t	age (days)
13	Harrodsburg (Limestone)	88	18	21	190
15	Louisville (Dolomitic Limestone)	72	12	8	223
	Neat Cement	45	22	14	285

$\bar{X}$ Mean value of the load to cause bond failure, in pounds

V Coefficient of Variation for the cores tested, in percent

M_t Number of cores

w/c Water-cement ratio

s/c Sand-cement ratio

VITA

VITA

Charles Frey Scholer was born in Manhattan, Kansas, on May 31, 1934. He received his primary and secondary education in public schools in Manhattan, Kansas, and was graduated from Manhattan High School in 1952.

Mr. Scholer received the BSCE degree from Kansas State University in 1956 and the MSCE degree from Purdue University in 1957. He was then employed by Burgwin and Martin, consulting engineers, in Topeka, Kansas, prior to entering the U. S. Air Force early in 1958.

Mr. Scholer had Air Force assignments in both Alaska and Texas as a first lieutenant and was released from the service early in 1960. He was then employed as assistant county engineer for Riley County, Kansas, until he returned to Purdue University in September of 1960. He has since served as a research assistant with the Joint Highway Research Project and as an instructor in the School of Civil Engineering.

Mr. Scholer is a member of the American Society of Civil Engineers, American Concrete Institute, Sigma Tau and Sigma Xi.